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Concrete Work on Sparkman St. Bridge, Nashville, Tenn.

By W. F. CREIGHTON.*

Davidson County, in the State of Tennessee, is now building two bridges across the Cumberland River at Nashville—one at Sparkman St. in South Nashville, the other at Jefferson St. in North Nashville. On account of the 100 ft. clearance at low water required by the United States government both bridges are very long and this length is made up necessarily in the approaches. Each bridge consists of three steel truss spans—a 175-ft. span on either side of a 318-ft. channel span—supported on concrete piers, and the Sparkman St. bridge is approached at one end by a reinforced-concrete bent and girder viaduct and at the other by a similar viaduct and two reinforced-concrete bowstring trusses. The total length of the Sparkman St. bridge is 3,260 ft. and of the Jefferson St. bridge, 3,070 ft.

The two bridges are very similar in their details, but this article will be devoted to the Sparkman St. structure, particularly to the concrete work there.

SPARKMAN ST. BRIDGE ACROSS THE CUMBERLAND RIVER, NASHVILLE, TENN.

Concrete Piers.

Fig. 2 shows details of a typical main pier which supports the steel spans. It will be noticed that it has been measurably lightened by the use of interior openings and by arching the top part.

Work was begun Aug. 22, 1907, on the cofferdam for the west channel pier of the bridge. The dam was the common crib type in three divisions; the outer sections were 8 ft. wide and were filled with sand and gravel; the middle section between cribs was 6 ft. wide and was filled with a mixture of sand, loam and gravel. The cribs were built of 4 × 6-in. timber and in sections 8 ft. long. These were started on the bank, floated to place, finished, and sunk; 1 × 6-in. lagging was nailed to the inside of the cribs to hold the puddling material. A 6 and a 10-in. centrifugal pump were set upon the dam and by Sept. 23 the water was low enough to allow excavation to begin. Solid rock was encountered at a depth of about 16 ft. below low water level, and having small channels across it which gave good anchorage, was in excellent shape to start the concrete. An additional precaution was taken by drilling a number of 1½-in. holes and placing in them dowels 12 ins. long.

Concreting was begun Oct. 12 on the footing and was carried up to the belt course, about 56 ft. high, in 4-ft. lifts with a derrick, and from there it was built with an elevator. The mixer used was a 17-ft. cube, and, for the work below, was stationed on the dam; but as high water was expected before the pier could be completed, the mixer and hoisting engine were stationed on a catamaran made by connecting two barges. This was anchored on the down-stream end of the pier and connected to the elevator frame by runways.

FORMS.—The forms were made of 1 × 3-in. tongue-and-groove flooring connected by 2 × 4-in. studding and in sections 4 ft. wide and varying length. A 4 × 8-in. wale belted the studdings at top and bottom and these were joined together through the pier by ⅝-in. rods. These rods were cut in three sections—two 14 ins. in length, or long enough to extend through the forms and the wale; the other piece was long enough to joint the short sections and they were connected to the two short outer lengths by cast-iron hexagonal nipples. The short pieces passed through the wales and lagging and screwed into the cast nipples about ½-in. When the outside nuts were removed it was possible to unscrew the 14-in. pieces and the long piece was left in the concrete.

This method was very satisfactory but expensive, and on the east channel pier a simpler and cheaper method was used. The wales were held together at the top by a 3 × 10-in. plank bolted to them; about 4 ins. below this a No. 8 wire was placed, joining the wales and twisted in the center. When the concrete had set the 3 × 10-in. piece was removed and used again above on the next section and the wire held the forms together at the bottom. Five 3 × 10-in. pieces were used in the length of the pier and at the ends of the cut water the wales were bolted together. The circular ends of the piers were built at first of circular forms made of narrow wooden strips, but it was found that heavy galvanized iron bent to the radius and nailed inside the forms made a better face and much less expensive.

CONCRETE.—Concrete was mixed in two classes, "A" and "B"; "A" concrete of 1:2:4 proportion, and "B," 1:3:6. Cement, limestone dust from ⅝-in. down, and stone from ⅝ to 1½-in. were the ingredients. This mixture makes an especially white concrete, which is, of course, due to the limestone dust. "A" concrete was used in a 12-in. strip around the outside of the piers and for all the reinforced work; "B" concrete was used in the interior of the piers, retaining walls, and some of the column foundations. Concrete was mixed very wet and frequently the men could stand knee deep in the mass. Tools to face the concrete made of ⅝-in. rods with a 2 × ¼ × 4-in. blade were found to give the best results, though several different styles have been tried.

REINFORCEMENT.—Eight 1-in. plain round rods were used to reinforce the arches, and 1-in. plain round rods bent to fit were used to reinforce the coping, with additional rods ½ × 12

*With Foster & Creighton Co., Nashville, Tenn.

ins. placed 15 ins. on centers at right angles to the face to give additional reinforcement to the cornice. Steel woven fabric of 4-in. triangular mesh was placed near all faces below the belt course.

The bank piers were built along similar lines, and were founded on two towers or legs 10 × 14 ft. which extend to rock. Just below the surface of the ground the towers are joined by 2-ft. curtain walls, which gives the solid appearance. In the piers on the east side much difficulty was encountered in the excavation, due to the nature of the soil, which was a deposit consisting of clay above and quick sand below, and it was necessary to excavate to a depth of about 40 ft. to obtain rock. Heavy cribbing was necessary to withstand the earth pressure, and, on account of the wet sand, it was difficult to keep out the water encountered below water level of the river. In the construction of the cofferdam for the east channel pier it was decided to found the crib on rock and, instead of three divisions, to build a single crib 16 ft. wide; so the site of the dam was dredged and the crib constructed in much the same manner as the west one. The puddling material was a mixture of sand and gravel which was pumped from the river bottom into the crib. Pumps were set to work, but it was impossible to lower the water more than a few feet; and, as it was thought that the puddling material was faulty, a strip of 16-oz. canvas 20 ft. wide was stretched around the outside of the dam; the bottom of the canvas was pulled out from the dam by a diver and sacks filled with clay were placed on it to hold it down. The pumps, a 6-in. and two 10-in. centrifugals, were started and again failed to get the water out. From examination it was decided that the water was entering underneath the dam and through crevices in the rock. This theory was afterwards proved to be the correct one. As it was getting too late in the fall to adopt new measures, the work was abandoned for the winter. When the water was low enough this spring, steel sheet piling was driven around the outside of the dam and the water was pumped out with little difficulty.

The retaining walls at each end of the approaches were completed as soon as possible to allow the embankments to be built.

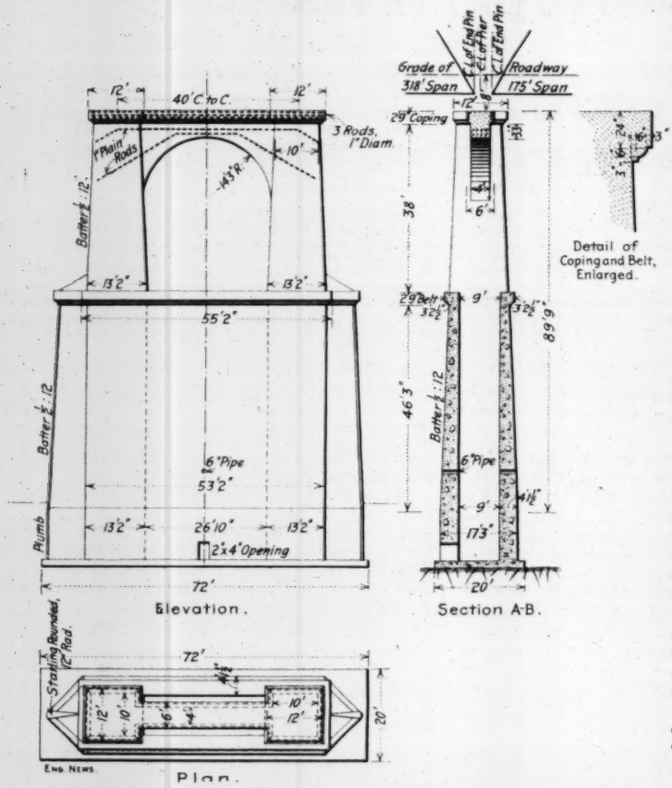

FIG. 2. TYPICAL CHANNEL PIER, CUMBERLAND RIVER BRIDGES.

At the east end of the bridge the wall is a winged abutment about 30 ft. high, which has broad footings, 15 ft. wide × 21 ins. thick, founded on clay, on which are built a narrow face wall 12 ins. wide at the top and 18 ins. at the bottom, supported by counterforts 2 ft. wide and 10 ft. centers, all heavily reinforced.

One interesting fact developed in the excavation for the footing for this retaining wall, which was through a yellow clay, was the large number of land crabs which were found to exist in the soil, and some fears were entertained that they might undermine the structure; but though the fill is almost completed no cracks have developed.

The retaining wall at the west end of the bridge is U-shaped and is 289 ft. 6 ins. long by 60 ft. 11 ins. wide, and begins at the present street grade at west end and rises on a 4.5% grade, so that it clears a street car track at the east end; it is founded on small piers 3 × 3 ft. and 2 × 3 ft., which extend to rock at the high end. The piers are joined together just below the surface of the ground with a reinforced footing and on this the wall rests, designed in the same manner as the east retaining wall.

REINFORCED - CONCRETE BOWSTRING TRUSSES.—The most interesting details of this work are six concrete trusses, which span the yards of the Tennessee Central R. R. They vary in span from 85 to 105 ft. and are clearly shown in Figs. 3 and 4. The bottom chord, which varies from 18 × 30 ins. in the shorter spans to 18 × 36 ins. in the longer, is an important part of the structure as it acts as a tie to take the horizontal components of the end thrusts; about 1/10 of its area is taken by rods and the concrete only acts as a stiffener and to protect the steel. The top chord extends from the shoe at each end and follows practically the line of pressure of the equilibrium polygon resultant from the dead load and is practically all in compression; for dead load only the pressure will be uniformly distributed over the cross section, the greatest pressure is 394 lbs. per sq. in.; if, in addition to this, there be a live load of 80 lbs. per sq. ft. over the roadway, the unit stress will be increased to 512 lbs. per sq. in. According to tests of columns of the length of the members of the 1:2:4 mixture, the average strength per sq. in. on columns 60 days old that for the assumed loading the factor of safety is over 3.

When an unsymmetrical live load of two 16-ton street cars passing on either side of the center truss, the no longer approximately coincides with of pressure, and a possible stress of lbs. per sq. in. may be found.

FALSE WORK.—The false work trusses was quite complicated, due to the railroad tracks, which are on a curve and are not parallel. It was built in two decks of the lower, consisting of twelve legs each, were placed to clear the tracks, on which lower deck were placed 8 × 16-in. stringers to carry the upper deck, the bents of which consisted of twelve legs each placed at right angles to the center line of the bridge. Fifteen 8 × 16-in. stringers rested on the caps to support the top deck, and each supported forms for the concrete stringers immediately above it. The false work was 4 ft. wider than the viaduct to allow runways for construction travelers.

FORMS.—A simple method of building the forms for the trusses was used, by which most of them were built in panels on the ground and placed in position. The bottoms were made of 2 × 6-in. shiplap connected by 2 × 4-in. battens; the sides were made of 1 × 3-in. flooring connected by battens. The sides of the bottom chord were extended up at the posts and cut to fit the fillet curve. The narrow sides of the posts were then built, placing the lagging horizontally, and at the top the fillets were formed where they joined the top chord by lengthening the lagging and circular ribs were fitted between the lines of posts and bottom of the top chord and shiplap nailed on it between tangent points. The remainder of the forms were built on the ground in panels and fitted between the tangent points of the curves.

REINFORCEMENT.—The ends of the trusses rest on steel shoes which consist of a bottom plate and back plate connected by five triangular rib plates resting on masonry plates. The bottom plate of the shoes at the west end of each truss has slotted holes to allow for expansion and contraction. Each bottom chord consists of eighteen rods varying in size from 1⅜ to 2¼ ins. and are in three lengths, connected together by turnbuckles and secured to the back plate with hexagonal nuts. The top chord is reinforced by eight 1¼-in. round rods—four in the bottom, two at the sides, and two on top—and are laced by ⅝-in. rods. The posts have six 1-in. rods laced by ¼-in. rods, 15 ins. c. to c., and are arranged at the bottom to act as hangers.

POURING THE CONCRETE.—Each truss was poured in a single day, which was possible, as the maximum yardage was only 70.7. Work was

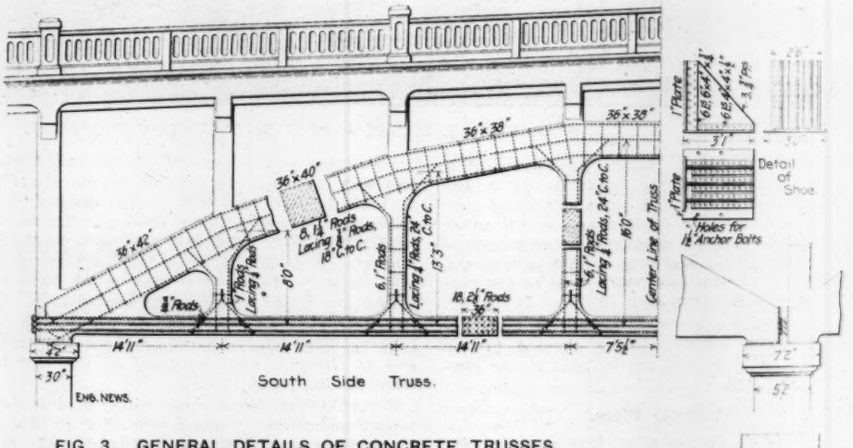

FIG. 3. GENERAL DETAILS OF CONCRETE TRUSSES.

started at both ends simultaneously and brought towards the center, all concrete being poured into the top chord and allowed to flow down the columns. When the top chord was filled it was covered with 2-in. flooring and heavily braced with struts from the 12 × 12-in. caps above. It was found necessary, in order to get the concrete to run to the bottom chord, to use hooks to pull it down; and it was never necessary to cover the bottom chord, though the fillet between the bottom chord and the posts were covered.

The trusses are braced laterally by 10 × 15-in. struts reinforced by four ¾-in. round rods. Centers were struck on the three trusses in each span after the last concrete had been in 30 days, and no deflection was noticeable with a level. Great care was used to strike the centers. The top chord was released first on all three trusses, then the bottom chord. The idea is to some extent a new one and is likely to be largely used in highway bridges, as it is possible to get a standard truss in which forms can be used repeatedly, and the yardage is slight and the cost not excessive.

Reinforced-Concrete Viaducts.

The viaducts consist of floor beams and stringers, with a 5-in. slab covering the top. (Fig. 5.) The sidewalk is 4 ins. thick and the entire surface is troweled to a template as it is laid in order to make it watertight. Over the trusses where the spans are about 15 ft. the floor beam is 16 ins. × 4 ft. 3 ins., with a cantilever bracket under the sidewalk. The stringers vary in size and there are 18 for each panel.

The standard viaduct panels are 30 ft. span supported by three columns, 24 × 24 ins. for the higher parts and 20 × 24 ins. for the lower, with the same number of stringers as the smaller sections. The rods were bent on the ground and assembled as a unit before being placed in the forms.

The footings for the columns are founded on clay for the most part, and where this is the case they are 8 × 8 × 1 ft. at the base, with a cap 3 ft. 6 ins. × 8 ft. × 1 ft., all of "A" concrete and heavily reinforced with 1 and 1¼-in. round rods. Above this extends a 2 ft. 6-in. × 2 ft. 6-in. cap of "B" concrete, without reinforcement, to the surface of the ground, where the reinforced columns begin. They are built of "A" concrete and are reinforced with four 1¼-in. rods in the corners with ¼-in. lacings, spaced 12 ins. on centers. The rods extend up into the floor beams.

The columns are built in towers of six columns each, joined by 10 × 15-in. struts reinforced by four ⅞-in. round rods for the longitudinal struts and four ¾-in. rods for the transverse struts, with stirrups averaging 2 ft. centers; 5/16-in. stirrups are used in joining the rods of the floor beams together; ½-in. rods are bent in a square C shape with the ends flared, and placed under the rods to hold them off the forms, and other spacers are placed between the different layers of rods and wired with No. 16 wire to hold them to place. The result gives a very solid unit.

The stringer rods, which consist of three straight and three trussed 1-in. corrugated rods, are built and placed in the same manner; the floor slab rods are ½-in. round and are spaced 4-in. centers over the center of the street and car tracks, 3½-in. centers on each side and 6-in. centers over the sidewalk, all running transverse to the center line. Twenty-three ½-in. bars are laid on these and run parallel to the center line. Wire chairs are used to space the floor slab rods and to hold them off the forms. We have found these to be perfectly successful.

The concrete, all class "A," was mixed wet and one panel of about 75 cu. yds. was poured in one day, and the sidewalks were poured the following day. The mixer was placed on the ground and the concrete raised by the travelers in bottom dump buckets and poured into the forms. The constructing travelers are supported on a track built on the ends of the false work bents and span the entire viaduct. They are used to erect the false work and place the concrete. When the trusses were being concreted the traveler was stationed in the middle of the truss and the concrete was poured from both ends up to a space 25 ft. wide, the length of the traveler, then the traveler was moved and the middle was poured.

Bulkheads were built in the center of each panel and the day's work stopped there.

The forms were built in a shop and are all of the collapsible style. Five panels of forms are used on each of the four sides of the river and are taken down and used over again.

No tests of the strength of the work have been made, but one accident happened which gave a test on the concrete hardly equaled. A piece of steel weighing 27,000 lbs. was dropped on the concrete sidewalk over a concrete truss. The piece was suspended by a chain in the middle and fell about 1 ft. The center landed about 2 ft. from the cantilevered end of the floor beam and on the sidewalk slab. The force of the blow was concentrated in center where the chain was hooked; the result was a hole in the sidewalk 6 ins. wide and 18 ins. long, with cracks extending out from it so that in repairing the place a slab 3 ft. wide and 8 ft. long was cut out. No cracks were made in the floor beam or in the concrete truss below.

The contract for both the Jefferson St. and the Sparkman St. bridges was let to the Foster & Creighton Co., of Nashville, Tenn., and the work is now being carried on by that company. The bridges were designed and are being built under the supervision of Mr. Howard M. Jones, Engineer for the Cumberland River Bridge Committee. It is expected that the two bridges, with the consequent land condemnation, will cost about $1,000,000.

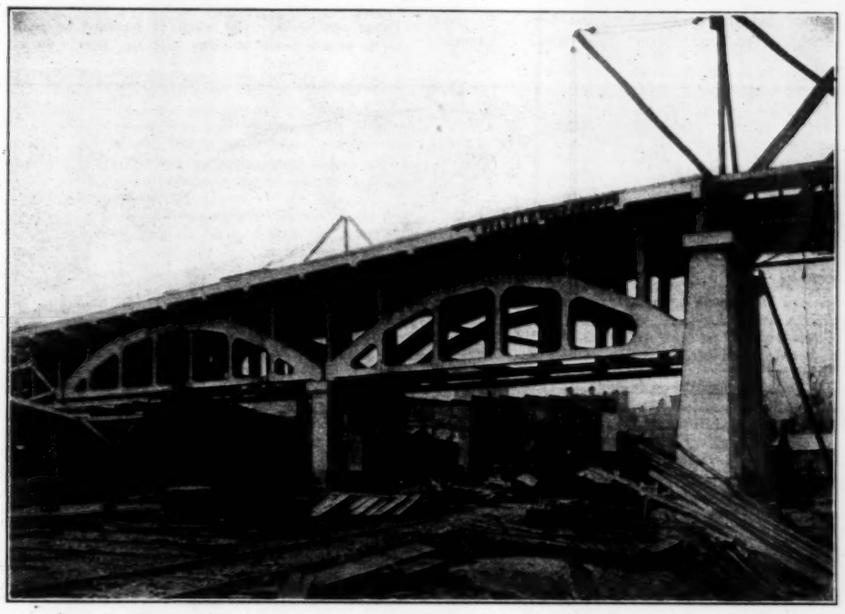

FIG. 4. VIEW OF CONCRETE TRUSSES, SPARKMAN ST. BRIDGE.

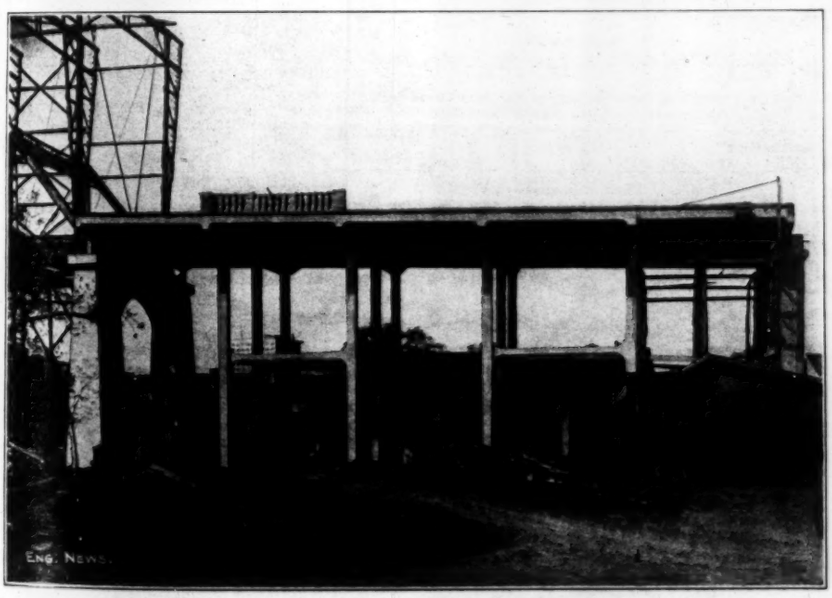

FIG. 5. VIEW OF PORTION OF VIADUCT APPROACH.

THE TOTAL PUBLIC DEBT OF THE UNITED States at the close of Dec. 31, 1908, was $1,316,851,168, excluding about an equal amount of certificates and Treasury notes offset by cash in the Treasury. The interest-bearing debt, however, was only $912,900,850. During January the interest-bearing debt was increased by $406,640. This made a total of $913,307,490, which was divided as follows by rates of interest: $730,872,130, at 2%; $63,945,460 at 3%, and $118,489,900 at 4%.

Description and Cost Estimates of the Proposed Georgian Bay Ship Canal.*

During the past few years the Department of Public Works of Canada has had under survey and study the project of a large-size ship canal to connect the upper of the Great Lakes to tidewater navigation on the St. Lawrence. It was decided that the route which presented the most desirable features was the one connecting Georgian Bay, the off-shoot of Lake Huron, with Montreal, via the French, Mattawa and Ottawa rivers and on this route, known as the Georgian Bay Ship Canal, detailed study has been concentrated. The surveys are now completed and the commission is about ready to make its final report, but pending some further investigation, what is termed an "interim report" has been presented which contains a description of the proposed canal and itemized estimates of the cost of the work. The text of the report is as follows:

*See Engineering News, Oct. 3, 1907, p. 371, for preliminary report on the canal surveys.

PROJECT.—The estimates presented herewith are based upon a project for a waterway at least 22 ft. in depth. It has been worked out in all its main details, which are shown on the plans prepared in this connection, and which will be explained very fully in the final report.

The style of navigation proposed is known as the "dam and lock system," with slack water reaches between structures. The whole is designed on such lines as to enable boats of large lake size (600 × 60 × 20 ft. draft) to pass from Lake Huron, through pond after pond to Montreal, the head of ocean navigation on the St. Lawrence River.

The project is essentially a river and lake canalization scheme, taking advantage of natural channels which fortunately can be made to form 80% of the distance from Georgian Bay to Montreal.

ROUTE.—(Fig. 1.) Of the 440 miles of projected navigation between the above mentioned points, from 410 to 420 miles follow the course of some river or lake.

For that part of the route from Georgian Bay to the height of land separating the watersheds of the Ottawa River and the Great Lakes, a distance of 81 miles, the French and Pickerel rivers and Lake [illegible] utilized. From Lake Nipissing, through [illegible] for a distance of 3½ miles, the route [illegible] waterway, with the exception of a [illegible] through which it is located.

This artificial cut leads into Trout L[illegible] Turtle Lake, the Little Mattawan River [illegible] which is utilized as far as Sand Bay at [illegible] a distance altogether of 21 miles. Trout [illegible] referred to above are very deep and [illegible] of water.

From Sand Bay there is a canal for three miles to the Mattawa River, which river is utilized as far as the town of Mattawa, a distance of 13 miles, where another canal cut ¾ miles in length makes an entrance into the Ottawa River.

This river, which expands into large and deep lakes in many places, is followed all the way down to the foot of Lake of Two Mountains (Oka Lake), a distance of 293 miles.

From the foot of Lake of Two Mountains to Montreal, a distance of 25 miles, either the St. Lawrence River or a branch of the Ottawa River called Rivière des Prairies, flowing north of the Island of Montreal, may be utilized.

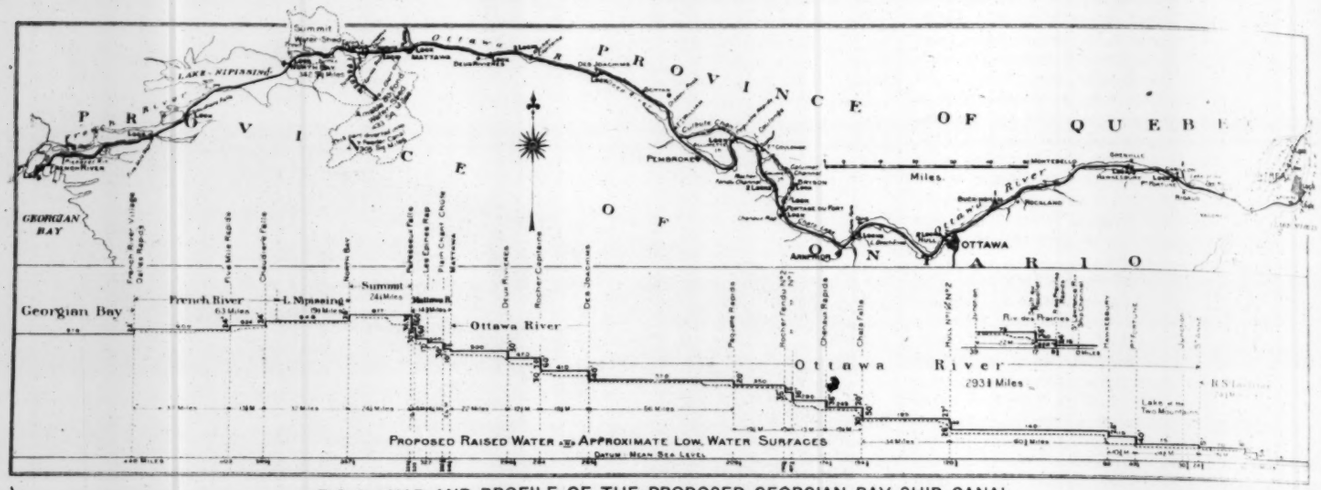

FIG. 1. MAP AND PROFILE OF THE PROPOSED GEORGIAN BAY SHIP CANAL.

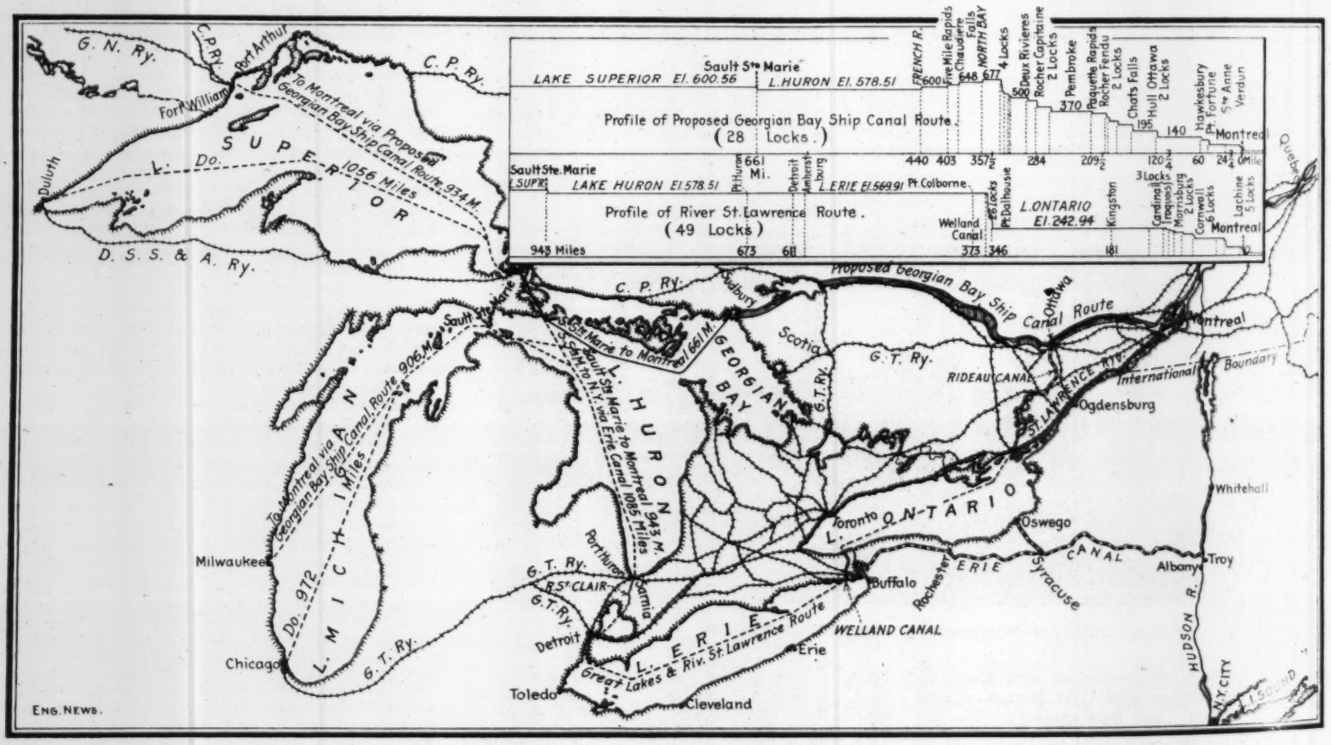

FIG. 2. MAP AND PROFILES OF PRINCIPAL ROUTES FROM THE GREAT LAKES TO MONTREAL.
(Showing Comparative Distances of the Various Routes.)

The former [illegible] has 5 miles of artificial waterway and the latter [illegible] 11 miles.

By the [illegible] route, the canal enters Montreal Harbor at its upper [illegible] By the second route the St. Lawrence ship channel [illegible] joined at Bout de l'Ile, some 11 miles below the [illegible] boundary of Montreal Harbor, or 17 miles below the City Custom House.

DISTANCE.—(Fig. 2.) Taking Port Arthur or Fort William [illegible] starting point, the distance to Montreal via the proposed [illegible] waterway, is 934 miles; via Lake Erie and the Welland [illegible] Canal, 1,216 miles; via Buffalo and Erie Canal to [illegible] York, 1,358 miles; giving a difference in favor of the [illegible] projected route of 282 miles as compared with the present St. Lawrence route, and of 424 miles as compared with the Buffalo-New York route.

Comparing the distance from Fort William to Liverpool via Montreal and via New York, we have:

	Miles.
Fort William to Liverpool via Georgian Bay canal..	4,123
Fort William to Liverpool via New York.....	4,929

giving a difference of 806 miles in favor of the Georgian Bay Ship Canal—Montreal route.

Other comparative distances can be found on the Transportation map (Fig. 2) prepared in connection with the report.

TIME OF TRANSIT.—This is affected by the length of restricted channels on the route, where speed has to be reduced, and by the number of lockages and consequent delays. A close computation of the speed allowable in the different stretches, with about three-quarters of an hour delay for passage at each lock, gives about 70 hours, as time of transit from Georgian Bay to Montreal.

With the advantage of shorter distance between terminal harbors, it is computed that the route will be from 1 to 1½ days faster than any other existing water route, under present conditions, from the head of the Great Lakes to an ocean port, apart from also having an enormous superiority as to carrying capacity. But as compared with a possible improved system of St. Lawrence canals to a depth of 22 ft., assuming that the number of locks would be greatly reduced and some of the channels widened, probably no practical benefit in time of transit could be claimed, the saving in distance being nearly offset by the longer stretches of lake and wide river navigation which exist through the Lake Erie and Lake Ontario route, where higher speeds would be permissible.

CHANNELS.—The total length of what may be termed canal cutting for the entire route is about 28 miles, by the project connecting with the St. Lawrence River above Montreal, through Lake St. Louis; and 34 miles, should the Riviere des Prairies route be selected.

The length of submerged channels to be excavated is about 66 miles, in stretches of varying lengths. Apart from this there is an aggregate of 14½ miles of route where obstructions such as shoals, sharp bends, etc., have only to be removed to form very wide channels.

Therefore, of the 440 miles constituting the waterway, 108 miles will require excavation work, for locks, approaches, canals, submerged channels, etc., leaving 332 miles of natural river or lake channels, which will not require any improvement beyond the raising of the water surface as recommended in connection with the project.

Taking into account the 14½ miles of obstructions, which after removal will leave wide free channels, the route may be subdivided as follows, in relation to width:

	Miles.
Canal cuts, 200 to 300 ft. wide, including necessary restrictions at locks.....	28
Improved channels, submerged sides, 300 ft. wide..	66
Free channels, 300 to 1,000 ft. wide and over.....	346
Total	440

The relative length of canals and submerged channels may be varied slightly, as it is an open question as to the exact point where the one ends and the other begins.

The sides of all submerged cuts will be shown by piers or clusters of piles at suitable distances to indicate the channel and to aid vessels in navigating. Along curves these piers will be provided with lights, and each different course will be defined by ranges.

The restricted channels are widened at all bends, and conditions for navigation in these restricted parts will be as good, it is expected, as on the St. Mary's River, or the St. Clair and Detroit River channels.

The depth of 22 ft. selected for the waterway will more than equal the conditions as they exist to-day in the channels connecting the waters of the Great Lakes, the St. Mary's River, Hay Lake, St. Clair Flats canal, and Detroit River.

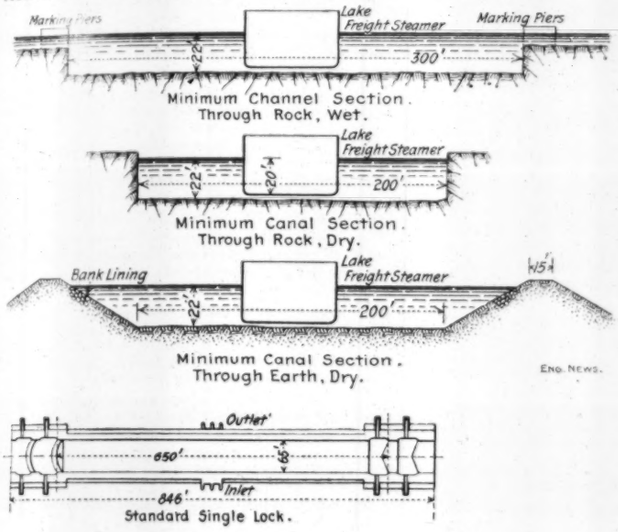

FIG. 3. STANDARD SECTIONS AND LOCKS ON THE PROPOSED GEORGIAN BAY SHIP CANAL.

The improvement carried out for these lake channels, since 1892, contemplated a depth of 20 ft. below the mean water surfaces as determined up to that time. Since then, however, the prevailing water levels of Lakes Huron, St. Clair and Erie have been almost continuously below the mean stage as formerly determined, and in consequence the actual draft available on account of lake fluctuations has been only 17 to 19 ft. (Report of Chief of Engineers, U. S. A., Vol. V., 1907.)

It has therefore been found necessary to increase this depth, and some of these channels are now being deepened to 21 and 22 ft. in order to obtain a safe 20-ft. draft at all times.

The Georgian Bay Ship Waterway, therefore, with a minimum depth of 22 ft., will compare favorably with any of the channels above mentioned, which govern the draft of boats on the Great Lakes.

The mileage of excavation in canals and channels for the route may be subdivided as follows, for each class of material encountered:

Rock, about	25 miles.	
Earth, about	13 "	
Mixed earth and rock, about.....	20 "	
		58 miles.
DRY EXCAVATION.		
Rock	18 miles.	
Earth	16 "	
Mixed earth and rock.....	16 "	
		50 miles.
Total		108 miles.

This mileage includes all points which are to be dredged or excavated, whether canal cuts, submerged channels or shoals. A small percentage of the excavation given as submarine rock work, might possibly be done in the dry, and the cost therefore reduced. In the estimates, when doubt existed, the rock excavation has been invariably classified as wet rock.

TERMINAL HARBORS.—As the harbor of Montreal forms the eastern terminus of the waterway, no special provision is made in the estimate for increased terminal facilities. By the time the waterway is completed, with the works now under construction and the extensive improvements proposed, the harbor will no doubt afford sufficient dockage facilities to meet the requirements of the increase in traffic contributed by the new route. As this traffic develops, facilities will be extended naturally as part of the harbor works.

The western entrance to the waterway on the Georgian Bay is formed by French River harbor. As this will be only a transit point to and from terminal harbors already established, no terminal facilities are required other than improvements in certain parts of the entrance, and increased aids to navigation. These improvements are included in the estimate.

SUMMIT.—The summit level embraces Lake Talon, the Little Mattawan River, Turtle and Trout lakes, their present surfaces being raised to elev. 677. Talon Lake will be raised 41 ft., and Trout and Turtle about 15 ft. above their present level. The locks at both ends of the Summit are designed to allow of the large lake thus created being lowered to elevation 671 without interfering with navigation. In fact, besides affording a wide and unobstructed route for shipping in transit, the lake will have two important duties; to absorb in part the excess in floods, and to store the reserves for the months of deficient water supply.

From the careful hydraulic investigations made, the available supply from the summit watershed, with the storage provided, will be 540 cu. ft. per sec. throughout the season of navigation, which will allow of an average of 24 passages per day or 5,040 passages for the season.

As the traffic develops, in the event of this supply being insufficient to meet the demand upon the Summit, the supply can be augmented by 700 cu. ft. per sec. by creating storage reserves at the head of the Amable du Fond River, and diverting it from its present outlet into the summit lake. This can be accomplished at an expenditure of $900,000.

These two sources of supply will more than meet the requirements at the Summit, should the waterway ever be worked to its full commercial capacity.

LENGTH OF NAVIGATION SEASON.—From the investigations made the opening and closing of navigation for the waterway would coincide to the opening and closing of ocean navigation for the harbor of Montreal, the length of the season being perhaps a few days shorter.

This would be governed by Lake Nipissing and conditions at the Summit and the Mattawa reaches, and the indications are that an average of 210 days would be available.

LOCKS.—The difference in elevation of 659 ft. between Montreal and the summit level, and of 99 ft. between the Summit and Georgian Bay is overcome by 27 locks ranging in lift from 5 to 50 ft. By the Riviere des Prairies route, however, this number is reduced to 26.

All locks are designed to be built of concrete. Regarding their size, lake boats have attained a length of over 600 ft., and the minimum dimensions of lock chambers should not be less than 650 ft. in length, by 65 ft. in width. The estimated cost of the locks is based on these dimensions, but in the final report the additional cost of building them 800 ft. in length by 75 ft. in width, should it be found desirable, will be given. In all cases the depth of water on the sills will be 22 ft. at extreme low stage. (Fig. 3.)

DAMS.—The navigation scheme requires the building of 45 dams of various sizes, not including those which will be required in connection with a system of storage reservoirs.

Generally, where the quantity of water is much above the canal requirements, the rock fill type of dam has been adopted. Where, however, it is important to economize water for lockages, concrete dams have been designed. The estimate of cost is based on these types of dams, and the stop-log system of regulation sluices has been adopted throughout, with the exception of a few locations where Stoney sluices are deemed necessary.

STORAGE.—Intimately connected with the navigation scheme is the question of control of the flood waters of the Ottawa River. This would be of great benefit to navigation as well as to industries along the river depending on water power. It is intended to effect this by creating large storage reservoirs, so regulated that during flood season they will retain a portion of the surplus waters, which will be gradually released during low water periods. This question will be discussed in the report, but cannot be closed, as a complete solution of the problem will require more extended investigations than it has been possible to make so far.

WATER POWERS.—The present plans for the construction of the canal entirely alter the general features of the river. For the purpose of lockage, the falls are concentrated, and all of the small rapids obliterated. The dams built for navigation purposes, by concentrating the fall at one point, eliminate the greatest difficulty in the development of the water powers. In addition, the needs of navigation require the elimination of extreme high water by the construction of a system of storage reservoirs at the upper reaches of the Ottawa River and its main tributaries, the water stored to be released at low water period, thus increasing the average low flow for power purposes.

The data collected up to date shows that nearly 1,000,000 HP. can be secured along the Ottawa and French rivers by the improved method of development. It is doubtful if more than 150,000 HP. at minimum flow could be developed under present conditions.

This question of water powers is still being investigated, as some more data have to be collected.

It may be mentioned, however, that the Chaudiere

TABLE II.—SUMMARY OF ESTIMATED COST, BY REACHES.
(Via Lake St. Louis.)

	Mile.	Locks.	Dams and Regulation.	Channels.	Damages.	Totals.
Montreal Reach	0 to 5	$1,090,700	$64,000	$1,352,300	$1,352,000	$3,859,000
St. Louis Reach	5 " 25	1,093,000	12,200	11,070,800	377,000	12,553,000
Oka Reach	25 " 49	783,800	360,800	937,300	251,100	2,334,000
Pointe Fortune Reach	49 " 60	1,477,400	361,900	1,880,900	140,600	3,860,800
Ottawa Reach	60 " 121	989,600	207,800	3,750,900	1,221,500	6,169,800
Hull Reach	121 " 122	929,700	5,700	730,400	658,000	2,323,800
Aylmer Reach	122 " 154	673,700	406,600	2,938,800	1,580,000	5,599,100
Arnprior Reach	154 " 174	818,200	477,700	1,421,400	28,300	2,745,600
Portage du Fort Reach	174 " 187	919,400	596,200	383,900	132,800	2,032,300
Rocher Fendu Reach	187 " 190	1,071,800	352,400	49,400	8,200	1,481,800
Coulonge Reach	190 " 209	1,052,300	588,500	2,294,900	4,700	3,940,400
Pembroke Reach	209 " 265	1,030,800	209,600	2,984,500	175,300	4,400,200
Des Joachims Reach	265 " 284	1,419,123	87,895	1,198,365	20,200	2,725,583
Rocher Capitaine Reach	284 " 296	2,028,428	495,383	1,394,089	.....	3,917,900
Deux Rivières Reach	296 " 318	965,058	452,447	870,516	182,400	2,470,421
Mattawa Reach	318 " 320	874,357	171,321	321,087	138,760	1,505,525
Plain Chant Reach	320 " 326	1,055,906	265,908	131,778	.....	1,453,592
Les Epines Reach	326 " 331	1,035,528	91,349	253,063	.....	1,379,940
Lower Paresseux Reach	331 " 333	1,825,642	184,383	513,111	.....	2,523,136
Summit Reach	333 " 358	2,659,852	290,536	5,403,079	20,000	8,373,467
Nipissing Reach	358 " 390	810,306	81,219	2,147,732	263,010	3,302,267
Five Mile Rapid Reach	390 " 403	932,306	69,766	2,160,781	.....	3,162,853
Pickerel Rapid Reach	403 " 442	1,440,020	223,926	4,517,678	330,000	6,511,624
		$26,977,926	$6,057,533	$48,706,379	$6,883,870	$88,626,108
		30%	7%	55%	8%	

Construction of locks, dams, channels, piers, lighting, damages	$88,626,108
Contingencies, engineering, administration, say 10%	8,862,892
Storage of flood waters and regulation basins, telephones, etc.	2,200,000
Total	$99,489,000
Feeder at Summit, when required	$900,000

powers are not interfered with by the project. In the case of undeveloped water powers which are destroyed, and which have been leased or sold by the Ontario or Quebec governments, a certain sum has been placed in the estimate to cover possible claims. No doubt, in many cases, it will be possible to compensate the claimants by giving them power privileges at some of the dams built in connection with the project.

The final report will give all the information available regarding this question.

DAMAGES.—On several of the reaches considerable land will be flooded permanently. Most of this land is now every year inundated from four to six weeks. The area so flooded has been computed for each reach, and the amount to be paid, included in the estimate at a fair value per acre. In the case of damage to buildings, the cost of their removal to higher ground or purchase has been considered and provided for.

CONSTRUCTION.—A careful analysis of the work to be performed shows that it would take from three to five years to develop all contracts and place the whole route under active construction. Some of the sections where heavy submarine excavation is encountered would require at least five years to complete, under the best conditions of labor and equipment. It may be fairly stated, therefore, that a period of ten years from inception, would be necessary to open the waterway to navigation. This would mean an average expenditure of about $10,000,000 per year.

UNIT PRICES.—For the various items that appear in the estimate of cost, the prices were adopted after careful consideration, and they conform generally to the prices paid by the department for similar work. These prices have been altered, where necessary, to meet special conditions of location and character of the work to be performed.

Respectfully submitted,
A. ST. LAURENT,
Asst. Chief Engineer and Engineer-in-Charge.
C. R. COUTLEE,
District Engineer.
S. J. CHAPLEAU,
District Engineer.

Approved,
ENGENE D. LAFLEUR,
Chief Engineer.

As noted in the above the canal is divided into 23 reaches, for each one of which a separate estimate has been made. However, it will be sufficient to give here the two summarized estimates for the alternative projects.

TABLE I.
Route A.

Via Montreal, Lake St. Louis, Ste. Anne de Bellevue, Ottawa, Rocher Fendu channel, Coulonge, Pembroke, Des Poachims, Mattawa, Talon Lake, North Bay, Lake Nipissing and French River.

Locks, dams, channels, piers, lighting, damages	$88,626,108
Contingencies, engineering, administration, say 10%	8,862,892
Storage of flood waters, regulation basins, telephones, etc.	2,200,000
Total	$99,689,000
Feeder at Summit, when required	$900,000

Route B.

Same as Route A, excepting that Riviere des Prairies, north of Montreal Island, is followed instead of Lake St. Louis and St. Lawrence River above Montreal.

Locks, dams, channels, piers, lighting, damages	$83,354,508
Contingencies, engineering, administration, say 10%	8,335,492
Storage of flood waters, regulation basins, telephones, etc.	2,200,000
Total	$93,890,000
Feeder at Summit, when required	$900,000

Note.—Land damages are partly covered by estimation and partly by contingencies. In most cases of undeveloped water powers, it has been assumed that owners could be compensated by being granted power privileges at nearest dam. Cost of damages, at best, cannot be well defined. In ten years from now, it is likely that damages to pay would be much larger, as conditions on the river would be much more involved. This amount cannot be well foreseen. It might be larger than estimated by one or two millions according to conditions at the time of construction and legal view taken of some of the claims.

The cost of "Route A" tabulated under the various reaches is given in Table II under various items in Table III.

In Fig. 3 is given the various sections of the canal and the plan layout of the ty[illegible] lock.

TABLE III.—SUMMARY OF ESTIMATE OF COST, BY ITEMS.
Route A.

Description.	Quantity.	Unit prices.	Cost.
Rock excavation, submarine	cu. yds. 8,322,554	$1.50, $3.00 and $3.50	$23,982,780
Rock excavation, dry	" " 18,574,496	$1.00 and $1.50	19,587,750
Earth excavation, dredging	" " 8,935,667	20, 25, 30 and 35 cts.	2,140,073
Earth excavation, dry	" " 10,836,537	20, 30, 35 and 45 cts.	3,223,690
Concrete in dams, locks and entrance piers	" " 1,841,259	$7.50	13,810,493
Concrete in dams, second class	" " 60,698	4.50	273,700
Granite Masonry in locks	" " 2,474	50.00	123,700
Rock fill work (dams, embankments and behind cribs and walls)	" " 7,880,652	0.50	3,940,337
Rock fill work (riprap and borrow pit)	" " 510,088	1.00	510,088
Bank lining	" " 114,300	2.00	228,600
Earth fill (dams, embankments and back of walls)	" " 3,770,078	5, 10, 15, 25, 40, 50 and 60 cts.	984,229
Cribworks	" " 1,910,102	$3.00 and $3.50	6,191,405
Timber mattress	" " 84,583	$1.35	114,186
Stop-logs, regulating works and machinery			1,849,680
Unwatering			354,976
Lock gates (steel)			2,610,106
Equipment and power			875,920
Bridges, railway crossings			1,462,882
Lighthouses, guide piers, lighting			879,734
Damages to land and water supply powers, drainage, railway and highway diversions, etc.			5,482,340
Total			$88,626,108
Contingencies, engineering, administration, say 10%			8,862,892
Storage of flood waters, regulation basins, telephones, etc.			2,200,000
Total			$99,689,000
Feeder at Summit, when required			$900,000

Tests of Piers Built of Sand-L[illegible] Brick and Common Brick.*

By H. H. SCOFIELD.†

The material for this paper was taken [illegible] phases of six seniors in the School of Civil Engine[illegible] Purdue University in 1906 and 1907.‡ The tests made at the laboratory for testing materials, and [illegible] were prepared under the direction of Profs. W [illegible] and W. K. Hatt. The tests extended over [illegible] of two school years and consisted of the careful [illegible] storage and testing of 32 brick piers or [illegible] columns. Minor tests were also made of the stren[illegible] the individual bricks and of the mortar from w[illegible] the piers were made.

The piers were built by an expert brick[illegible]on under methods as nearly as possible representing [illegible]mon practice. Each pier was finished at each end [illegible] a square cement mortar cap 3 ins. thick. These [illegible]ent caps served the purpose of giving square, smo[illegible] ends as bearing surfaces in the testing machine, [illegible] also an even distribution of the load over the cro[illegible]section of the pier. The piers, which were about 4 [illegible] high and 12 ins. square, were made up of courses with [illegible] bricks to a course.

The mortar used was common lime mortar, proportioned 1 to 3½, and was laid approximately ½-in. thick. All voids were carefully filled with mortar so that the final pier was a solid mass. Each individual brick was dipped in water before being laid, so that it would absorb no water from the mortar. Samples of the mortar were taken from the various piers as they were made and molded into test pieces for later tests in tension and compression.

Some of both kinds of piers were stored out of doors exposed to all weathers from December, 1906, to the spring of 1907. A part of both kinds were stored in the laboratory and were thus protected from any extreme changes of temperature and moisture. Specimens of the individual brick were also stored under the same conditions for later tests in flexure and compression.

The piers were tested in compression in a 300,000-lb. Riehle testing machine. They were bedded in the machine in plaster of paris, which was allowed to set hard before tests were begun. Pressure was brought upon the pier through a universal spherical pressure plate, thus insuring axial loading and an even distribution of load. Readings were taken of the load at the first observed crack of the specimen and of the maximum load the pier would stand. Readings were also taken of the compression of a length of 3 ft. of the column.

The strength of the individual brick was obtained from tests of full-sized specimens. The size of brick was approximately 2¼ × 4 × 8¼ ins. The strength of the mortar was obtained from tests of 2-in. cubes in compression and of 1-in. square briquettes in tension.

[The general results are shown in the accompanying table, compiled from the tables given with the paper.—Ed.]

TESTS OF SAND-LIME AND COMMON BRICK.

1.—Compression Tests on Brick Piers.

	Sand-Lime Bricks, tons per sq. ft.		Common Bricks, tons per sq. ft.	
	1st Crack.	Max.	1st Crack.	Max.
Age about 60 days.				
Piers exposed to weather	14.6	19.9	12.9	23.9
Piers not exposed	12.2	24.2	13.1	22.3
Age about 450 days.				
Piers exposed to weather	18.3	23.7	29.0	34.0
Piers not exposed	20.3	25.9	30.1	32.9

2.—Tests of Individual Bricks.

	Cross-bending: Mod. of rupture, lbs. per sq. in.		Compressive strength, lbs. per sq. in.	
	Sand-lime.	Common.	Sand-lime	Common.
Bricks exposed to weather	231	788	1,240	3,247
Bricks not exposed	334	818	2,0[illegible]	3,336

3.—Tests of Mortar.

The lime mortar used (1:3½), with an average age of 55 days, showed a tensile strength of 39.7 lbs. per sq. in., and a compressive strength of 321 lbs. per sq. in.

AN ARCHITECT'S LICENSE BILL is pending before the Indiana state legislature. It provides for licensing architects and plumbers after examination by a commission to be appointed by the Governor. A $25 application fee is fixed in the bill.

*Abstract of a paper presented at the annual meeting of the Indiana Engineering Society at Indianapolis in January.

†Instructor in the Laboratory for Testing Materials, Purdue University, Lafayette, Ind.

‡M. E. Burrell, C. A. Neal, W. E. Webb, A. P. Williams (1906); P. M. Tebbs, H. T. Frushour (190[illegible])

The Theory of Eccentrically-Loaded Columns, Including the Influence of Shear Distortion.

By HENRY S. PRICHARD.*

In response to the evident desire and urgent need of engineers for additional information regarding steel columns, the Government authorities have begun a series of tests. As the capacity of the testing-machine to be used is only 800,000 lbs. it will not be possible to establish direct criteria for even moderately large columns. What ever benefit engineers will derive from these tests must be largely through the insight afforded into the laws which determine the strength and behavior of members in compression.

Judged by the preliminary program outlined by the committee of engineers who are advising the Government, the series of tests will afford an excellent opportunity to verify the theory of columns as far as it has been developed, and the theory if confirmed by the tests, will in turn so complement the experiments as to give their results the widest possible application. For this reason it is deemed advisable to call attention to the theory of eccentrically loaded columns; to improve it by including the influence of shear on deflection; to develop it farther by making the analysis of the effect of eccentricity general for a combination of any eccentricity at one end with any eccentricity at the other (instead of restricting it to the case of equal eccentricity at both ends on the same side of the axis); to extend it by including an analysis of shear in columns; and to give the particular results for some special cases.

The influence of shear on deflection, and through deflection on strength, is quite important in very short columns with very weak lattice, but in ordinary cases such influence is small and the chief advantage of including it in the analysis is to make this fact evident. The analysis is so made that the influence of shear on deflection can readily be omitted from the critical equations whenever it appears to be negligible.

PROBLEM 1: To find the deflection due to bending moment of a given point A in the axis of a beam or column of constant cross-section, with reference to a line which is tangent to the axis at a second point B.

Let Δ_1 = the deflection due to bending moment.
- x_1 = the distance from point A to any point C in the axis between A and B.
- r_0 = the distance from the axis of the beam or column to any fiber of the cross-section C.
- M = the bending moment at the cross-section C.
- I = the moment of inertia of the cross-section C.
- E = the modulus of elasticity of the material.
- m = the moment of the area of the moment diagram between the given points, about the point whose deflection is sought.

From the theory of flexure we know that the stress intensity at any fiber of cross-section C is Mr_0/I. The elongation or compression of this fiber on the small length dx_1 is:

$$\frac{dx_1\, Mv_0}{E\, I}$$

If the bending in the beam extends from B to C only, the point A, whose deflection is sought, will be shifted from its original position A_1 (Fig. 1) to A_2. If, however, the small additional length dx_1 at C also bends, point A will move to position A_3, and the angle included between successive positions of the axis will be the same as the angle included between successive cross-sections. From similar triangles the deflection $A_3 A_2$ due only to the piece dx_1 is found to be

$$d\,\Delta_1 = \frac{Mx_1\, dx_1}{E\, I}$$

and by summing this expression for all the pieces dx composing the entire length $B\,A$ the whole deflection is obtained:

$$\Delta_1 = \Sigma_0^{AB} \frac{Mx\, dx}{E\, I} = \frac{m}{E\, I} \qquad (1)$$

The value of Δ_1 given by eq. (1) is in reality measured on a curved line, but for most cases in practice the curve is so slight that the deflection may be considered as measured perpendicular to the tangent.

PROBLEM 2: To find the deflection due to shear in a beam or column of constant cross-section with reference to a line joining points of no bending moment.

Bending in any part of a beam causes a change in the direction of the axis which is sustained through succeeding portions of the beam, as shown in Fig. 1, but the shear in any part, while it causes an offset in the axis, does not change the slope of the axis except in that part itself, as shown by the full lines in Fig. 2. (The broken lines show the deflection from bending moment alone.) This is evident from the fact that shear

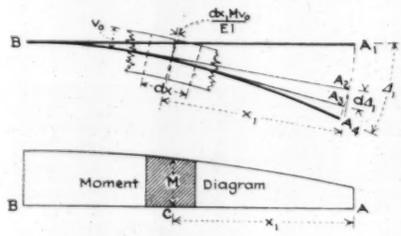

Fig. 1.

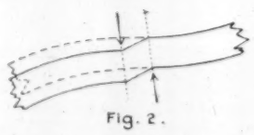

Fig. 2.

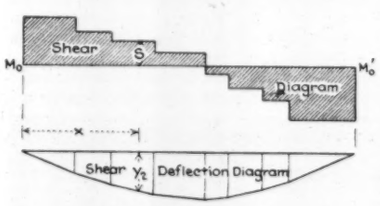

Fig. 3.

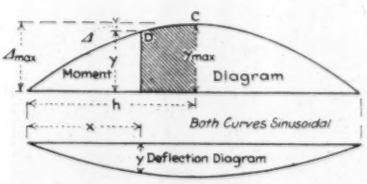

Fig. 4

between successive cross-sections, while it causes distortion of the intermediary material, does not alter their inclination.

Between points of zero bending moment the inclination of the axis of a beam or column under the action of shear will alter at each cross-section where the shear alters, as indicated in Fig. 3.

Let S = the shear,
- y_2 = the deflection due to shear, measured from a line joining points of no bending moment,
- t = the deflection due to a shear of unity in a length of unity,
- x = distance from a point of zero bending moment as an origin, to the cross-section whose shear-deflection is y_2.

Then the following relations hold:

$$dM = S\, dx.$$
$$dy_2 = S\, t\, dx.$$

For $x = 0$, $M = 0$ and $y_2 = 0$, hence, integrating,

$$y_2 = t\, M. \qquad (2)$$

PROBLEM 3: To find the deflection of a beam or a column of constant cross-section with reference to a line through points of no moment when the diagram of moments is a sinusoid.

Let y_1 = the bending-moment deflection.
- y_2 = the shear deflection.
- $y = y_1 + y_2$ = the total deflection of any point (D, Fig. 4), with reference to a line through points of no bending moment.
- x = the distance of point D from point of no bending moment.
- h = the distance from point of no moment to point of maximum moment (C, Fig. 4).

As the moment diagram is a sinusoid, the moment at any point D is

$$M = M_{max} \sin\left(\frac{\pi}{2}\frac{x}{h}\right)$$

The area of the moment diagram between C and D is

$$\Sigma_D^C M_{max} \sin\left(\frac{\pi}{2}\frac{x}{h}\right)dx = M_{max} \cos\left(\frac{\pi}{2}\frac{x}{h}\right)\frac{2\,h}{\pi}$$

If we again call m the moment of that part of the moment diagram between D and point of maximum moment C about the point D as center of moments, the variation of m for a small shift of point D equals area $C\,D$ multiplied by dx, or

$$dm = M_{max} \cos\left(\frac{\pi}{2}\frac{x}{h}\right)\frac{2\,h}{\pi}\,dx$$

and by summing this for the entire distance $C\,D$ the value of m is found:

$$m = \Sigma_D^C M_{max} \cos\left(\frac{\pi}{2}\frac{x}{h}\right)\frac{2h}{\pi}\,dx$$
$$= M_{max}\left[1 - \sin\left(\frac{\pi}{2}\frac{x}{h}\right)\right]\frac{4\,h^2}{\pi^2}$$

For $x = 0$, the value of Δ_1 obtained by substituting the value of m in eq. (1) is the deflection from the tangent at C, which is equal to the maximum value of y_1 measured upward from the base as in Fig. 4. For any other value of x, y_1 is the difference between this maximum ordinate and the value of Δ_1 for that point; in other words the bending-moment deflection is

$$y_1 = M_{max} \sin\left(\frac{\pi}{2}\frac{x}{h}\right)\frac{4\,h^2}{\pi^2\, E\, I} \qquad (3)$$

From eq. (2) the shear deflection is found to be

$$y_2 = M_{max} \sin\left(\frac{\pi}{2}\frac{x}{h}\right) t \qquad (4)$$

Therefore the total deflection is

$$y = y_1 + y_2 = M_{max} \sin\left(\frac{\pi}{2}\frac{x}{h}\right)\left(\frac{4\,h}{\pi^2\, E\, I} + t\right) \qquad (5)$$

In this case the curve of deflection, that is the elastic line, is similar to the curve of moments. As this is not a general characteristic of the elastic line, if in any particular problem we find that the curve of deflection of a beam or column of constant cross-section is similar to the curve of moments, this fact proves that both curves are sinusoids.

PROBLEM 4: To determine the conditions of stability in a centrally loaded ideal column of constant cross-section with pivoted ends.

Such columns do not actually exist, but an analysis of their stability is a convenient preliminary to further study.

Let p = the load per unit of cross-section,
- A = the cross-sectional area,
- r = the radius of gyration of the section,
- x = the distance to any point in the axis from a point of no moment (whether pivot or point of contraflexure), measured parallel to the line of thrust,
- y = the deviation of the axis from the line of thrust. In a centrally loaded column this is the deflection, as in Fig. 5.

The moment may be expressed as

$$M = p\, A\, y,$$

which shows that the curves of deflection and moments are similar, and hence are sinusoids, as explained in connection with eq. (5). The formulas of Problem 3 are therefore applicable. Substituting Ar^2 for I and $p\,A\,y$ for M_{max}. $\sin\left(\frac{\pi}{2}\frac{x}{h}\right)$

*1526 Frick Building, Pittsburg, Pa.

in eq. (5) and solving for p,

$$p_i = \frac{\pi^2 E}{\frac{4h^2}{r^2} + \pi^2 E A t} \qquad (6)$$

This is Euler's formula modified to include the influence of shear on stability, as seen by putting it in the form:

$$h = r\sqrt{\frac{\pi^2 E}{4p} - \frac{\pi^2 E A t}{4}} \qquad (7)$$

Whatever deflection is assumed, the same value for p results. Hence the load which will hold a centrally loaded ideal column in equilibrium in one deflected position will hold it in equilibrium in any deflected position within the elastic limit. This is termed a state of indifferent equilibrium. Under a larger load the column would fail; under a smaller load it would straighten.

Eq. (7) shows that for each cross-section and for each possible centrally applied load within the elastic limit there is a certain limiting length between pivots or points of no moment ($2h$), below which, if the conditions are ideal, there can be no deflection, and beyond which the column will buckle.

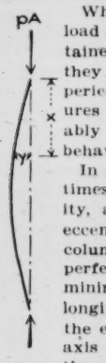

Fig.5.

While these conclusions as to limiting load and length can be approximately sustained by very careful laboratory tests, they are not in accord with ordinary experience for the reason that the departures from ideal conditions which inevitably occur in practice cause columns to behave in a radically different manner.

In practice, loads on columns are sometimes applied with intentional eccentricity, and always with some unintentional eccentricity in reference to the axis of the column. Moreover, the true axis is not perfectly straight. However, for determining the deflection and the maximum longitudinal stresses (but not the shears) the effect of initial curves or kinks in the axis can be approximated by assuming the column to have a straight axis and the load to be applied with a certain equivalent eccentricity. Thus the problem may be reduced to the case of a straight column under eccentrically applied load.

It will be shown in what follows that an analysis of eccentrically loaded ideal columns can be based on the elastic line of the centrally loaded column. The chief difficulty in dealing with the problem is not one of theory, but lies in the fact that the unintentional eccentricity depends on variable and uncertain conditions of practice. The assumptions which should be made in this regard are matters for investigation, experiment, experience and judgment.

If instead of a single centrally loaded column a continuous line of equal centrally loaded columns is conceived as held in deflected position, as in Fig. 6, the pivots will be points of contraflexure, and the condition of the successive columns will differ only as regards the direction of the deflection, which will alternate from one side of the line of thrust to the other. By selecting a suitable portion of this multiple column, it is possible to duplicate exactly the conditions as to stress, shear and deviation of axis from line of thrust, of any eccentrically loaded column, whether the eccentricity be at one or both ends, or whether it be on the same or opposite sides of the axis.

PROBLEM 5: To determine the relations between the load, stresses and deflections in the general case of an eccentrically loaded column of constant cross-section; the load may be applied eccentrically at one or both ends of the column and on the same or opposite sides of the axis, as in Figs 7a–d.

Let v = the distance from the axis to the outer fiber of the cross-section.

e = the eccentricity or ordinate of a known point in the axis, measured from the line of thrust.

u = the distance of this known point from a point of contraflexure, measured parallel to the line of thrust.

i = the angle between the line of thrust and the axis of the column in its unstrained state.

G = the perpendicular distance, at a point of contraflexure, from the line of thrust to the axis of the column in its unstrained state.

f = the stress in the outer fiber at any cross-section.

Δ = the deflection perpendicular to the line of thrust of any point in the axis of a column with reference to the position of the said point when the column was unstrained.

D = the deviation of the axis from the line of thrust at point of maximum moment in a continuous line of centrally loaded columns (= $y_{max.}$).

As the elastic line is a sinusoid, the conditions sketched in Fig. 7 show that

$$y = e\frac{\sin\left(\frac{\pi}{2}\frac{x}{h}\right)}{\sin\left(\frac{\pi}{2}\frac{u}{h}\right)} \qquad (8)$$

Hence

$$y_{max} = D = \frac{e}{\sin\left(\frac{\pi}{2}\frac{u}{h}\right)} = e\,\mathrm{cosec}\left(\frac{\pi}{2}\frac{u}{h}\right) \qquad (9)$$

From the theory of flexure, the stress in the outer fiber from bending is $Mv/Ar^2 = pyv/r^2$, from which the total stress in the outer fiber is

$$f = p \pm \frac{p\,y\,v}{r^2} \qquad (10)$$

By referring to Fig. 7 it will be seen that

$$\Delta = y \mp G \pm x\tan i \qquad (11)$$

From these general equations (8, 9, 10 and 11) any special case can be developed. The one which follows is chosen on account of its importance and to connect this analysis with those of other authors.

PROBLEM 6: To determine the relations between the load, stresses and deflections in a column of constant cross-section when the load is applied to each end with equal eccentricity and on the same side of the line of thrust (Fig. 7a).

Let l = the length of the column; then

$$u = h - \frac{l}{2} \qquad (12)$$

which, substituted in eq. (9) gives

$$D = e\,\mathrm{cosec}\left(\frac{\pi}{2}\frac{h-l/2}{h}\right) = e\sec\left(\frac{\pi}{2}\frac{l}{2h}\right) \qquad (13)$$

For this case $G = e$ and $\tan i = 0$; hence

$$\Delta_{max} = e\sec\left(\frac{\pi}{2}\frac{l}{2h}\right) - e \qquad (14)$$

The maximum compression is, by eq. (10),

$$f_{max} = p + \frac{p\,e\,v}{r^2}\sec\left(\frac{\pi}{2}\frac{l}{2h}\right) \qquad (15)$$

and the load in terms of the maximum stress is

$$p = \frac{f_{max}}{1 + \frac{e\,v}{r^2}\sec\left(\frac{\pi}{2}\frac{l}{2h}\right)} \qquad (16)$$

If the value of h is taken from eq. (7), after first omitting the influence of the shear by taking $t = 0$, and substituted in eq. (16), it gives

$$p = \frac{f_{max}}{1 + \frac{e\,v}{r^2}\sec\frac{l}{2r}\sqrt{\frac{p}{E}}} \qquad (16a)$$

This equation (16a) was given by Mr. Marston in his discussion of Prof. Cain's paper "Theory of the Ideal Column,"* and has been independently developed by other authors. A close approximation was given by Prof. Cain in July, 1887,† and similar approximations have been devised by others.

*Trans. Am. Soc. C. E., June, 1897, Vol. XXXIX., p. 108.

†Journal of the Franklin Institute, July, 1887.

PROBLEM 7: To determine the ... in an eccentrically loaded column of cor... cross-section. The load may be applied ... trically at one or both ends of the colum... to the same or opposite side of the axis.

Let S = the transverse shear at an ... tion.

S_e = the transverse shear at a ... whose ordinate is e and ...

S_m = the transverse shear at a ... traflexure.

α = the angle which a tangent t... axis at any point mak... line of thrust.

$$\mathrm{Tan}\,\alpha = \frac{dy}{dx} = \frac{\pi D}{2h}\cos\left(\frac{\pi}{2}\right.$$

The direct compression in the colum...

$$\frac{p\,l}{\cos\alpha}$$

The shear S, therefore, is

S = direct compression × sin α = ...

$$= p\,l\tan\alpha$$

and inserting the above value of tan α,

$$S = \frac{p\,A\,\pi\,D}{2h}\cos\left(\frac{\pi}{2}\frac{x}{h}\right) \qquad (17)$$

From this equation it appears that the curve of shears is a curve of cosines. The maximum shear, therefore, occurs where x is zero, i. e., at a point of contraflexure, so that

$$S_m = \frac{p\,A\,\pi\,D}{2h}$$

The shear at any other point (u, e) in terms of its own coordinates (independently of x and D) is found from eq. (17) by putting x equal to u and eliminating D by substituting its value as given by eq. (9); then

$$S_e = \frac{p\,A\,\pi\,e}{2h}\cot\left(\frac{\pi}{2}\frac{u}{h}\right) \qquad (18)$$

PROBLEM 8: To determine the stresses, shears and deflections when part of the cross-sectional area has a lower modulus of elasticity than the rest.

Some authorities claim (and judging from special tests it is probably a fact) that the true modulus of elasticity of steel is an almost constant quantity, even for different grades, although the extensions and compressions recorded in ordinary tests indicate a wide range. Unfortunately, however, the shortening of material in compression under the first application of the load is sometimes much greater than indicated by the "true modulus," even under stress intensities well within the yield point, and this greater shortening affects the stiffness of the column, especially if it is greater in one leaf or side of the column than in the other.

Fig.6.

While it is highly probable, as claimed, that the true modulus would obtain the second time the material is strained, it is not certain that a column designed for perfect but actually having defective elasticity will survive the first application of the maximum load, and, if it does survive, it may have a permanent deflection to weaken its resistance to subsequent loads. It is wise, therefore, to consider the possibility of a lower modulus in one side or leaf than in the other. Prof. Fidler, in discussing this matter in a paper on the Practical Strength of Columns before the Institution of Civil Engineers in 1886, adopted a range from 23 to 29 millions for the modulus of wrought iron. Extensive experiments in this regard are a desideratum.

The following means of allowing for any variations in elasticity may occasionally be useful.

Suppose a part of the cross-section, area a_1, has the modulus of elasticity E_1, different from the modulus E pertaining to the rest of the cross-section. Consider the area a_1 replaced by an

area a_1 E_1/E with a modulus of elasticity E like the rest of the column. With this altered area determine the position of the neutral axis, moment of inertia, radius of gyration, stresses, shears and deflections as for an ordinary column of homogeneous material, but multiply the stress and shear intensities obtained in the substituted area by the factor E_1/E to obtain the intensities of the stress and shear in the actual area. If the stress intensity in the actual area is limited to the value f, the limiting stress intensity in the substituted area should be $f\ E/E_1$.

PROBLEM 9: To determine the deflections, stresses and shears when the load is centrally applied at the ends of a column of constant cross-section which, in its unstrained state, has its axis slightly curved to a sinusoid with ordinates varying from zero at the ends to e_c at the center.

For these conditions the length of the column, l, is equal to $2\ h$.

For convenience, let q, which may be termed a factor of buckling, be substituted for p, and l for $2\ h$ in eq. (6), that is,

$$q = \frac{\pi^2\ E}{\frac{l^2}{r^2} + \pi^2 E\ A\ t} \qquad (19)$$

By a process of reasoning similar to that employed in solving previous problems it can be shown that the elastic line is a sinusoid and the following equations can be developed:

$$\Delta_{max} = e_c \frac{p}{q-p} \qquad (20)$$

For maximum compression,

$$f_{max} = p + \frac{p\ e_c\ v}{r^2}\left(1 + \frac{p}{q-p}\right) \qquad (21)$$

$$S_m = \frac{p\ A\ \pi\ e_c}{l}\left(1 + \frac{p}{q-p}\right) \qquad (22)$$

While the foregoing analysis, by recognizing the influence of shear, rests on a more refined

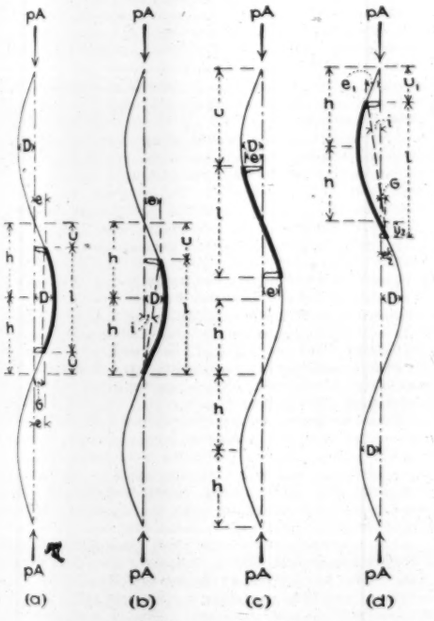

Fig. 7.

basis, and is more comprehensive in its scope, than other analyses with which the writer is familiar, it is in harmony with the work of those who during the last twenty-five years have abandoned the false trail made by Tredgold, Gordon and Rankine, and have made a fresh start from the classic formula which Euler presented to the Berlin Academy in 1759. The mathematical analyses of this school of investigators have disappointed the vain hopes of those who desire a simple universal formula by which the strength of any column can be accurately determined when its length, cross-section and details of its end connections are known, but they have helped to demonstrate the fact that the very nature of the problem involves considerable uncertainty in most of the cases which arise in practice. Further, to the engineer who can form a sound judgment as to the imperfections in material, workmanship and construction to be expected in practice, these analyses of themselves furnish a guide for designing columns which leads to better and safer results than a blind following of tests.

While the critical equations in the above analysis may occasionally be of direct use in analyzing very important or unusual cases, they are too complicated for direct application in ordinary practice. They can be made very useful, however, in indicating laws in a practical way by applying them to series of hypothetical columns varying progressively with regard to certain conditions, and the results obtained, in connection with the results of tests, can be made the basis for examining existing, or, if, desirable, in devising new simple rules for ordinary cases. A study of good column practice in the light of theory, tests and experience is reserved for a future occasion.

Street Railways vs. People.*

By SAMUEL H. BARKER.†

"Set the table over again." That was the succinct and meaning answer of Peter A. B. Widener, when asked in the early days of the Philadelphia Rapid Transit Co. how the promoters could escape paying up in full on the $30,000,000 of stock, which had been issued 10% paid.

But the easy scheme contemplated for reaping profits and then shifting to others responsibility for assessments has failed to work out as intended. The explanation is simple. After participating in many carefully prepared feasts, the invited guests—otherwise investors—have finally declined to come to further repasts.

This applies generally. Now there has befallen in many of the big cities of the United States the trouble which is entailed by the breaking down of any system, even though it be a vicious one. The public is suffering from want of adequate and good street railway transportation. This is true in many cities, while in some the situation is intolerable.

Prospective earnings of street railways have been discounted to the immense profit of a comparatively few promoter-capitalists. In this system of gathering up small properties and then merging them, at enhanced valuations, into new companies, no one man has been more proficient than Mr. Widener. While the idea did not originate with him, but with William H. Kemble, now many years dead, Peter A. B. Widener, long intimately associated with William L. Elkins, has been apt in putting it to practical use. Thomas F. Ryan and others have been mere followers. The system of exploiting the public through the ready and long-bearing medium of street railway transportation, made hugely profitable by monopoly-conferring franchises, must be regarded as a Philadelphia product.

Right here, understand that street railway capitalization and management upon lines which have now led to trouble, if not actual disaster, did not always nor invariably contemplate a fraud upon investors. The old horse cars earned big profits. Electric railways promised, and, in fact, have yielded, far greater returns.‡ Upon this theory and a calculated increase in population relied upon to expand traffic, was based the since heavily overworked and frequently abused scheme of recapitalizing street railway properties at intervals of a few years.

When electricity came to supplant horse power, street railway interests, acting largely in the dark, and, as it has been proved, in real ignorance, thought to earn dollars instead of cents. They did not know the cost of street railway operation and maintenance with electricity as power, but they did understand and exploit to the limit, the possibilities of creating and selling stocks and bonds. As promoters, these organizers of street railway companies knew their business and profited hugely. As practical street railway managers they have abundantly demonstrated themselves utter failures.

Before electricity had been many years in use evidence began to accumulate that street railway interests had miscalculated. The greatly quicker service had vastly increased traffic and largely extended the area of population. Time, not feet, measures the distance people can live from their daily work. Longer rides, which the electric railways first encouraged as a sure way to develop traffic, obviously resulted in lowering the average per mile fare. Transfers were freely granted, also upon the theory that anything which would stimulate travel must result in benefit to the transportation company.

Then it was found that for economy of operation the small cars and light tracks first provided had to be replaced by larger and heavier types. Some shrewd managers saw what confronted the electric railways and met the situation both intelligently and honestly. They saw that to keep on capitalizing new expenditures, made essentially to replace an obsolete system, meant financial ruin in the end.

Among the first to appreciate this, and the first to have the courage to put into effect a system of street railway accounting which squares with financial honesty, was C. G. Goodrich, Vice-President of the Twin City Rapid Transit Co., of St. Paul and Minneapolis. The example of this company, set ten years ago, is now forcing itself on the American Street & Interurban Railways Association. The fast and loose methods of accounting which have been in vogue with street railways, and used in a forlorn attempt to bolster up excessive capitalizations and to produce book profits large enough to meet exorbitant rentals and guarantees, must be discarded for an honest system.

What the Twin City Rapid Transit Co. has done has been to replace its entire system since 1897 without adding materially to its capital account. This policy has kept fixed charges low, these aggregating in fact only 15% of the gross earnings. With the Philadelphia Rapid Transit Co., whose fixed charges and guaranteed rentals reach a maximum figure, about 43% of the gross earnings are required to meet the fixed charges.

With its moderate fixed charges the Twin City Rapid Transit Co. is easily able to charge to current operating cost all expenditures for maintenance and renewals, and also to provide a depreciation fund which will pay for ultimate replacements of power plant and other equipment. Besides all this the company pays full dividends on both its preferred and common stock.

This true policy in electric railway management and its comfortable results are exceptional. The promoter interests, as might be expected, took the easier course, adopted the "set the table over again" plan, and made new companies to guarantee those previously organized, conveniently forgetting the difference between earnings and capital.

Now, all at once, from one end of the country to the other, has come the day of reckoning. Street railway companies must meet the difficult, and, for the security holders, the trying problem of squaring themselves with necessity and the follies and crookedness of the past. This is a big problem. On the one hand are the electric railways of the United States, with their huge aggregate capitalization of over $4,000,000,000, and the large, yet relatively small number of investors who hold the stocks and bonds.

Standing on the other side are the 86,000,000 American people, now asked to pay for electric railway transportation enough to support in prosperity corporations too often conceived in iniquity, financed without regard to principles of honesty or the safety of investors, and managed in utter disregard of the rights of the public. The issue is too big to be settled by any appeal to sentiment.

Electric railway cars are operated in large cities at small cost. The public has never been instructed on this point—the basic one in any adequate understanding of the subject—but it has been told and re-told about the wonderful service it gets—the fine cars, the long rides, the quick time—all for an absurdly low fare!

Official statements of companies operating street railways in the big cities of this country show that it costs, according to varying conditions, anywhere from about 12 to nearly 18 cts. per car mile to perform the service. That is, to operate a car one mile and to cover all expenses up to the point of interest on the capital investment—to pay for maintenance of tracks and equipment, for the electric power required and the wages of men—all that is necessary is for the car to carry three to four passengers for each mile run. All passengers over this small number make profit riders. Their fares go to pay interest and dividends.

Take the Philadelphia Rapid Transit Co., which reports the lowest operating cost shown by any big city street railway. This company handles more than 500,000,000 passengers a year, or an average of 1,400,000 per day. For the fiscal year ended June 30, last, 12.06 cts. was the average cost per car mile.

Taking the car mile cost of street railway operation in the big cities of the country, the average is close to 15 cts. It follows that $1.50 would be the total expense of operating a car over an average route of ten miles. Therefore, 30 passengers carried during such a run and each paying a nickel fare, yield sufficient revenue to meet the entire operating and maintenance cost of the service rendered. Every additional passenger produces net profit towards interest and dividends.

Having thus seen the cost of street railway service per car mile, let us next observe the earnings. The strap-hanging multitudes doubtless will conclude that such earnings are greatly larger than they really are, forgetting that length of rides and light travel at certain times

*From an address before the Wharton School of Finance and Commerce of the University of Pennsylvania.

†Financial Editor of the Philadelphia "North American."

‡[Greater returns per mile of road, but the profits on the capital actually invested have often been less.—Ed. Eng. News.]

and on some lines are factors which bring down the average receipts.

Still, these earnings per car mile are not low when taken in connection with the expenses. They range from about 22½ to 30 cts. per car mile, with the cost of the service varying, as already stated, from about 12 to nearly 18 cts. Taking an average of actual figures for nine large cities we get car mile earnings of 25.4 cts., or $2.54 for a ten-mile run, to set against an operating cost of about $1.50. This amounts to a profit of nearly 40% on the gross business done.

Turning to actual figures, the gross earnings per car mile of street railway systems in various cities, and the percentages of profit which these receipts yield the operating companies, are as follows:

	Car mile earnings.	Profit on business.
Philadelphia	22.54 cts.	46.5%
Detroit	22.63 "	38.2%
Cleveland	22.81 "	35.3%
Toronto	23.5 "	46.8%
Buffalo	26.0 "	38.4%
Milwaukee	26.2 "	47.7%
Boston	26.8 "	33.2%
Pittsburg	28.1 "	37.0%
St. Paul-Minneapolis	29.99 "	42.5%

Few business operations produce larger profits than these. The United States Steel Corporation, commonly, and not without reason, regarded as the most effective of trusts, last year earned a total manufacturing profit of under 25½% on its gross business. For 1907 the Pennsylvania Railroad system, which handles the greatest transportation business in the world, showed net earnings from operation equal to not quite 20% of its gross receipts.

What, then, means the bankruptcy and disintegration of the New York City railway system, the present serious trouble of the Philadelphia system, and the far and wide cry of street railway interests for higher fares? Only that outrageously heavy capitalization and exorbitant fixed charges are now overtaxing an abundant earning power.

Let us examine this matter. Over-capitalization on a stupendous and wicked scale is what has carried down the New York structure. That, and inordinately high rentals and guarantees piled one upon another is what has done the work. And now the people of New York City suffer. They are offered a disrupted street railway service and asked to pay double for it.

When the Interborough-Metropolitan Co. was launched early in 1906 as the last stroke in the consolidation and capitalization of New York City transportation facilities, more than $95,000,000 of absolute water was injected into the proposition. Than what was done in this case no better illustration could be found of the method of creating securities, so-called, against other securities at an agreed-upon valuation.

The Interborough-Metropolitan Co. was created to acquire and hold stocks of the Interborough Rapid Transit, Metropolitan Securities and Metropolitan Street Railway companies. Substantially the entire capital stock of each of these three corporations was taken over. Exactly what happened in the transaction is this:

Stocks acquired.	Securities given,	
$33,912,800 Interborough Rapid	Bonds at $200 per share	$67,825,600
	Common stock at $99 per share	30,521,520
46,059,600 Metropolitan Ry.	Preferred stock at $100 per share	46,059,600
	Common stock at $55 per share	25,332,780
$29,392,000 Metropolitan Securities on which 75%, or 22,044,000 was actually paid in, common stock at $93.50 per share		27,481,520
$102,016,400		$197,221,020

Thus came into existence $95,204,620 of the final tremendous capitalization of the New York City railway system. It had no more basis than arbitrary valuations set upon the securities taken over. The Widener-Belmont-Ryan group of promoters simply sat down together and agreed among themselves what the new company, which they were about to organize, should give for the securities of the other companies, which they controlled.

Through a devious process of financial jugglery there was built up a top-heavy corporation structure which it was sought to make the New York street railway system carry. In the end, the Interborough-Metropolitan system had a net outstanding capitalization of $520,000,000 in round figures. Think of it!

Last fall this outrageously constructed fabric ripped. Various of the merged companies went into receivership. One after another guaranteed rentals have been defaulted. Up to the end of June last there had been default on $144,000,000 of securities, upon which the annual interest and rental charges had been nearly $8,000,000.

Take one other illustration of rank over-capitalization of street railways. The Philadelphia Rapid Transit system has a net outstanding capitalization of $148,000,000. Of this amount about $44,500,000 is water. In other words, that is the total difference between what was actually paid in and the par of securities issued.

Of the $30,000,000 capital of the Union Traction Co. only $10,500,000 is paid in, yet the Philadelphia Rapid Transit Co. guarantees 6% dividends on the full par value of the stock. In an earlier deal the Union Traction Co. gave $14,535,940 for Electric Traction stock, on which only $8,297,920 had been paid and $15,200,000 for People's Traction stock, on which $6,000,000 had been paid.

Such methods of street railway capitalization are justified by the perpetrators and beneficiaries on the theory that in order to finance the development of the properties it was necessary to give large inducements to capital. But this is specious argument, and is disproved by practice and experience elsewhere.

Note, then, the different system pursued in Boston, where the inherited hard but true Puritan ideals would not tolerate and have not permitted that utter defiance of canons of right and honesty, which, until very recently, has been successfully dared in connection with street railway exploitation in nearly every important city of the United States.

Not to the extent of a copper cent does water exist in the capitalization of the Boston electric railway system, neither in the surface lines, the elevated system, nor the leased city-built subways, all of which systems are operated by the Boston Elevated Railway Co. Instead, the securities outstanding represent cash received by the issuing companies to an aggregate amount of $4,202,668 in actual excess of the face value of the securities issued.

That is a record of financing upon lines quite different from those elsewhere commonly followed. What is more, it stands forward as a knock-down argument for the excuses which are offered for the other practice of putting out street railway securities based not upon value existing or created, but upon unrealized hopes of future earnings.

Also does the Boston situation overthrow the contention of promoter-capitalists that without great profits in sight money would not be forthcoming for street railway development. Big capitalists may figure on big turns, but small investors are ready to put money into securities of assured value. Nearly 12,000 people own the less than $31,000,000 Boston electric railways' stock.

Note now the relative capitalization per mile of track of the surface line street railway system in ten chief cities:

New York	$251,500 per mile
Philadelphia	239,000 " "
St. Louis	215,000 " "
Buffalo	125,000 " "
Milwaukee	125,000 " "
St. Paul	112,500 " "
Toronto	100,000 " "
Cincinnati	93,200 " "
Cleveland	90,000 " "
Boston	74,000 " "

Thus we find a range in street railway capitalization from the reasonable amount of $74,000 per mile for the prosperous and superbly operated Boston system, up to the outrageously excessive amount of over $250,000 per mile for the hopelessly bankrupt New York system.

It must be obvious that excessive capitalization and extortionate fixed charges and guarantees, rather than unremunerative fares and low profits on the business done, make the present troubles of some street railways. Under such a load, the New York street railway combination has staggered to a fall and the dishonest financial structure has collapsed.

Clinging still to a hope of being yet able to force value into millions upon millions of watered securities, the Metropolitan-New York City Railway interests boldly cut off transfers and seek to compel the public to pay two and sometimes three fares for the same service that formerly was rendered for five cents. The Public Service Commission has intervened to protect the people.

Not alone in New York have fares been raised. The Philadelphia Rapid Transit Co. is now getting fully $1,200,000 a year greater receipts as a result of various changes made since last May in its ticket and operating systems. Not satisfied with this, the proposition now is to establish a straight 5-cent fare. This would give the company fully $5,000,000 a year more. The management and those interested in the securities of the company say this is necessary, that unless the Philadelphia Rapid Transit Co. can get greatly larger income it cannot live and perform good service. Probably this is true with the tremendous capitalization, nearly one-third water, and the excessive rentals guaranteed. The amount which the company is paying to security holders equals nearly 8% on the cash capital actually invested. Certainly that is a good return, and all the public ought to be asked to pay. If the fund is so distributed that it yields no dividend on Philadelphia Rapid Transit stock, the reason is that excessive rentals of underlying companies eat up an undue share.

Not one of the street railway companies which has been carried beyond its depth by big capitalization and rentals too great for earnings to meet, has either intelligently or honestly presented its situation. There have been all sorts of attempts to mislead th[illegible] upon the people higher transportation ch[illegible]

Given facts and true conditions the A[illegible] may be trusted to deal fairly and libe[illegible] debtors. It is not the spirit of this fr[illegible] exact the pound of flesh. But they wan[illegible] square deal.

No sane man expects to get transport[illegible] cost of the service provided. Every ho[illegible] gladly pay as much as is honestly requ[illegible] the service and to yield a good income retu[illegible] ital actually invested and at the business [illegible] terprise.

This does not mean that the public will[illegible] street car fares such as are necessary to r[illegible] dividends upon fictitious capitalization. W[illegible] honesty is applied on both sides a dr[illegible] flood of financial water will flow away. T[illegible] settlement plan is not quixotic. It has be[illegible] in practical form.

In Chicago they applied the process. T[illegible] railway interest fought, and cried, and b[illegible] said it was confiscation and that the Amer[illegible] rights of property was subverted. The cour[illegible] that the public had rights in the matter. F[illegible] $50,000,000 of water was squeezed out of [illegible] panies, and that almost entirely out of the Ch[illegible] Traction Co., which was an outgrowth of [illegible] table over again" system. Under the settle[illegible] took form in the franchise contract of Feb[illegible] street railway properties were allowed a [illegible] $50,000,000.

Upon this initial valuation, plus the actu[illegible] of rehabilitating the properties and of additio[illegible] perty, the city allows the companies to take [illegible] out of their earnings over operation and m[illegible] The expenditures made are capitalized, for [illegible] tion, with 10% added for construction profit [illegible] allowed for financing.

All net earnings of the companies over and above the 5% allowed on the investment value, go 55% to the city of Chicago and 45% to the companies. Since the new arrangement has been in effect the city has received in cash about $1,600,000 as its share of the profits of its big street railways.

Chicago holds the right to buy the properties of the street railway companies at the agreed investment value at any six-months period during the life of the twenty-year franchises.

Under the franchise agreement, fares and transfers are fixed with the idea of guaranteeing the people continuous rides for a single fare to their destination anywhere within the wide limits of Chicago. There are various other provisions, among which is one for continuous expansion of the street railway system, all to be carried out according to plans furnished by the Board of Supervising Engineers.

That capital is not scared away from investment in Chicago street railways because the return upon investments made, the capitalization and even the operation of the system, is prescribed in the franchise contract with the city, is demonstrated by the fact that by the end of this year $20,000,000 of new money will have been put into the properties since the present arrangement was made.

Again, in Cleveland we find another striking example of readjustment of street railway troubles. Settlement of the problem in this case was made on the basis of three-cent fares and free transfers and the scaling down of capitalization to fit the value of the street railways as this was ascertained by expert appraisal. This took away 45%, or $10,530,000, of the capitalization of the old Cleveland Electric Co.

Upon the appraised valuation, which took into account unexpired franchise values as well as tracks, cars and power plants, the old properties were taken over by a new company, which in turn was leased by the Municipal Traction Co., which has a nominal capital of $10,000 and was not intended to make profits, but to put into street railway service all it earns over the agreed rental of the Cleveland Electric Co. Bad times helped to upset the working out of Tom L. Johnson's plan, but, at least, his efforts have given an honest capitalization.

There has been one other serious attempt to meet the street railway problem fairly. This is in Philadelphia, where, more than a year ago a sort of partnership agreement was made between the city and the Philadelphia Rapid Transit Co. This was designed, in large measure, to make it impossible to further exploit the "set the table over again" idea in Philadelphia.

It also aimed to get for the people good and adequate service. The company was obliged to call all the unpaid balance on its stock, and was barred from issuing new securities or changing fares without formal consent of the city. On its part, the city repealed various old ordinances which the street railway interests held prevented them from getting new capital necessary for extension of the system.

Any profits which the Philadelphia Rapid Transit Co. earns over 6% on its capital stock, cumulative from Jan. 1, 1907, go half to the city, and similarly a sink-

ing fund is ...e provided to enable the city to buy in the Rapid ...nsit franchises and property after June 30, 1957, for ...,000,000. In this arrangement the municipality ... ...y condoned the past capitalization of the Philadelphia ... ...et railway system. But it did not undertake to ... ...rwrite this at the expense of the people, should the ... ...ings of the Rapid Transit Co. on the then existing ... ...are basis fail to carry the water and heavy guarantees ... ...s which the corporation had shouldered.

Truly may ... ...be said that the street railway situation presented today in many cities of the United States is one in which ... the sins of the fathers are being visited upon the children.

Cold-Storage Warehouses of Reinforced-Concrete Construction.*

By J. P. H. PERRY.†

The purpose of this paper is to present the different methods of insulating buildings for cold storage or similar purposes and to give also an idea of the adaptability of reinforced concrete to this type of structure. Up to the present time there has been practically nothing written on the use of reinforced concrete in connection with insulation and there are undoubtedly many promoters of cold storage plants who have no proper conception of the advantages offered by concrete for buildings of this kind.

As the necessity for cold storage plants has developed, attention has been given the insulation question and the materials which would fulfill the requirements have been studied. It was soon apparent that insulating materials must possess the following properties: Low conductivity, minimum capillary attraction, low affinity for moisture, fireproofness, and minimum tendency to continue combustion internally after the external flame is extinguished. In addition to the above requirements the insulating substance must not be such as to pack down readily and leave air space at the top of the filled compartment. Furthermore, the insulation should be durable, cheap and easily installed.

In considering insulating materials it must be understood that their worst enemy is atmospheric moisture which must be kept out of the construction to obtain permanent satisfactory results. In cold storage work artificial conditions are set up. Difference in temperature means difference in pressure and as long as the refrigeration is maintained the warm atmosphere is pressing in through the walls everywhere. The resistant material must be such as to resist moisture rather than receive it.

In the early days of artificial refrigeration, which dates back not much over 40 years, it was the almost universal belief that the most effective insulation consisted of air spaces. It was recognized that the easiest way to prevent the circulation of the air was by filling the air spaces with some substance which would break it up. Thus in the primitive storage plants and in ice houses the walls were insulated by erecting studs and sheeting both sides to produce an inner and an outer wall and filling the space between with shavings or sawdust. Floors and ceilings were protected in the same manner, the filling material being packed between the joist. When it was realized that such a deadener (as the filling material is known) would soon collect moisture and lose nearly all its efficiency besides settling and leaving free air spaces, the breaking up of air spaces was effected by means of "boards and air spaces" formed by alternate sets of furring strips and matched boarding in several layers as the temperatures to be maintained required.

The next step forward in the development of the science of insulation was the use of mineral wool. This material is made by subjecting molten furnace slag to a steam jet. It is used for a filler the same as sawdust and possesses many advantages over the earlier materials. It is fireproof and does not tend to pack to the same extent.

Soon after the introduction of mineral wool, use was made of granulated cork. As a deadener it was light in weight, of low conductivity, and with very slight capillary attraction. It possessed some tendency to settle and leave free air spaces.

As the study of the cold storage problem progressed a radical change was brought about in the theory and practice of the insulation installation. Up until about ten years ago dead air was the basis of all insulation. It became evident that air spaces even though filled with deadening materials left a great deal to be desired in matters of efficiency, durability, and safety. The air in the spaces was not dead; instead, it circulated through voids in the filling material and in so doing carried moisture to all parts of the insulation. Even more important in throwing the balance against this method of protection was its non-fireproof qualities. It would be difficult to imagine a more grave fire risk than wooden walls filled with sawdust or shavings. Brick buildings insulated along the same lines, were proved serious fire hazards. So much so in fact that the insurance companies were forced in their own protection to make high rates on cold storage plants so numerous were the bad fires in this type of building.

The modern theory of successful insulation is based upon entrapped air; air that is actually dead. This means that the air must be in sealed spaces and that the spaces must be minutely small—otherwise the essential idea of "still air" is overthrown. The insulation must depend for its entrapped air upon the interstices or cells of the material employed rather than upon alternate layers of solid insulation and air spaces.

This new principle of insulation which does away with double walls with air spaces between, has brought into vogue solid insulation. In this type the protection consists of layers or blocks of various materials set in Portland or pitch cement directly against brick or concrete walls and protected on the inside surface by plaster or tile. The space occupied by solid insulation is much less than that utilized by the older construction. If a fireproof or slow-burning material be used in making the insulation, the fire risk is greatly reduced.

The materials available for this so-called "solid" method of insulation are chiefly cork and compressed mineral wool blocks called "lith."

Cork, by virtue of its natural state and the apparent use for which nature intended it, lends itself admirably to insulating purposes. In cold storage work, cork is efficient as a non-conductor of heat, is free from capillary attraction, and has no tendency to absorb moisture freely. When used as cork boards it is granulated, compressed in iron molds, and baked while under pressure, its own natural gum being liquefied by the heat and uniting the granules into solid blocks. These have fair structural strength and are quite fireproof.

Lith is made from rock fiber mineral wool and degummed flax fiber. In the process of manufacturing the settling propensities of the mineral wool are removed and the product given structural strength by the degummed flax fiber. The finished material is a strong, light, durable board. It consists of many minute cells which confine air with the least possible chance for circulation. Lith is fairly fireproof, a good non-conductor of heat, and compares when cost is considered, favorably with cork board.

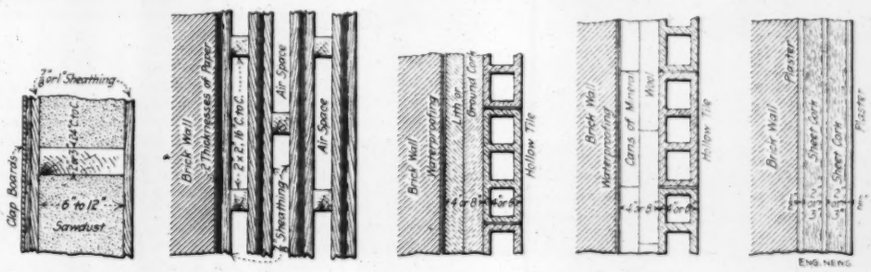

Fig. 1. Horizontal section showing side board and sawdust.

Fig. 2. Horizontal section showing boards with waterproofing paper.

Fig. 3. Vertical section showing hollow tile (plain).

Fig. 4. Vertical section showing hollow tile with insulation in galvanized iron cans.

Fig. 5. Vertical section showing solid insulation.

FIGS. 1 TO 5. METHOD OF INSULATING THE WALLS OF COLD STORAGE BUILDINGS.

Methods of Insulation.

In discussing the practical application of insulation the different methods will be considered in detail in their order of use; taking up first the wall construction, then considering the floors and roofs.

One of the earliest forms of insulation to be generally adopted is shown in Fig. 1. It is still in frequent use, particularly for natural ice storage houses. Briefly, the insulation consists of constructing a hollow wall with joists about 3 × 12 ins., spaced 24 ins. on centers and sheathed on both sides with ⅞-in. or 1-in. boards. Sometimes these are clapboarded on the outside to protect the air space from the weather. Occasionally this is not thought necessary. The space between the two sheathings is filled with shavings or sawdust, most often the latter. Although this form of insulation can be installed cheaply and rapidly, it constitutes a grave fire risk and owing to the tendency of the sawdust packing to absorb moisture rapidly it is not permanent.

Another method which is similar and yet a little more scientific and which is used to a considerable extent by one of the largest firms of meat packers in this country is shown in Fig. 2. A brick (or concrete) exterior wall is constructed of standard thickness. Against this are placed two layers of waterproofing paper, then ⅞-in. sheathing, preferably tongue and groove. A 2-in. air space is then formed by 2 × 2-in. furring strips about 16 ins. on centers. On these is nailed another layer of ⅞-in. sheathing against which are placed two thicknesses of waterproof paper. Then another thickness of ⅞-in. sheathing and a second air space is formed similar to the first. The alternation of waterproof paper, sheathing, air spaces and sheathing is continued to the thickness deemed sufficient to hold the storage rooms at the desired temperature. This insulation can be installed cheaply and in some cases has been found to be efficient. The chief point against it is the fire risk it creates. Its low initial cost is the main argument in its favor.

The structural details commonly employed in connection with installations of mineral wool show in Fig. 3. The interior bracing wall is generally of plaster blocks or hollow tile laid in cement mortar. Owing to the tendency of the packing to settle, unusual care has to be exercised in placing, to prevent arch action.

One way of utilizing the advantages of mineral wool and at the same time getting away from its settling has been worked out by a large Boston cold storage company. The details show in Fig. 4. In this installation galvanized iron cans are constructed 2 ins. in thickness and just large enough to hold a sheet of mineral wool. These after being filled are soldered tight and placed in the wall as in any insulating, the cans breaking joints and all joints being sealed with Portland cement mortar. In this manner the mineral wool is protected from moisture and also from settling. It would seem, however, that there must be a loss through the cans where they join. It is nevertheless stated that this form of insulation is very efficient and that it has given satisfaction. Its cost is somewhat higher than the other mineral wool installations.

The structural features employed when ground (or granulated) cork is used are the same as are shown in Fig. 3. The exterior wall is built of concrete or brick or terra cotta and damp proofed with any one of the legion of waterproofing processes. At a distance dependent upon the amount of insulation required an inner wall of concrete or tile is put up and the intervening space filled with ground cork. This form of insulation gives efficiency and has considerable durability. Its cost rather exceeds mineral wool. The chief disadvantages are its settling tendencies and its readiness to run out of any opening accidentally made in the supporting walls.

Coming now to solid insulation the structural details are covered in the main by Fig. 5. As already indicated, either sheet cork or lith may be used. Against the outer wall of standard construction is placed a sheet of cork 2 or 3 ins. thick set in Portland cement about ½-in. thick. Against this first layer of cork the second is applied with another ½-in. cement coat. The joints between the cork boards are broken both horizontally and vertically. The inner surface of the insulation is given a ½-in. plaster coat with sanitary finish. Generally the inside of the building wall is given a waterproof coat before the first cement binder is applied. This system is very efficient, is not attacked by moisture, is very durable, and generally speaking fireproof. It occupies minimum space. Its disadvantage is its high initial cost.

The question of interior walls and partitions in so far as their insulation is concerned does not present any important differences from the exterior wall protection except that the two sides of the insulating material are of necessity both of tile or block or thin concrete. In the ordinary case the temperatures on the opposite sides of interior partitions do not range as wide apart as with the enclosing walls of the building. The importance of properly protecting stair and elevator wells with insulated partitions will be discussed later.

Structural Design.

The design of the structural portions of cold storage buildings is of prime importance. The usual type of construction having walls and floors joining is subject to serious loss at these points. The most recent practice in artificial refrigeration buildings erects the frame of the structure independent from the enclosing walls, which latter are made entirely self-supporting and separate from all floor beams and columns. This scheme increases the efficiency of the insulation sufficiently to make it worth while. Fig. 6 gives roughly the main features to be obtained in this design. The curtain wall being self-supporting permits of the carrying of the insulation by the face of the interior columns, beams, girders and slabs, thereby securing continuous insulation. The exterior or curtain wall would be tied to the structural frame by steel anchors connecting at the floor lines. To

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†With the Turner Construction Co., 11 Broadway, New York City.

eliminate as far as possible a loss through these ties, they should be wrapped with hair felt or similar material. In any event their combined cross sectional areas would constitute but a very small fraction of 1% of the total wall surface and their conductivity value would, as compared to the loss through I-beams resting on bearing walls, be at a minimum.

The problem of insulating the conventional type of building has been solved as far as the elemental conditions will allow by the method shown in Fig. 7. This undoubtedly is the lowest in cost. The insulation runs from floor to floor stopping at the underside of the beams and the floor arches. The beams and floor of course carry on into the wall and cannot be protected. Another and better method is to construct floors only up to the inner line of the insulating partition. (See Fig. 8.) The beams and girders rest on the bearing walls or on the wall columns or wall girders with consequent loss, but the insulation prevents the entire floor section from conducting to the walls.

As to whether or not the floors of a cold-storage building should be insulated is an open question. Some plants

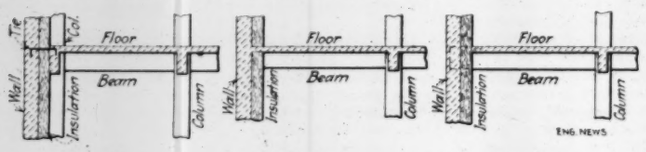

Fig. 6. Fig. 7. Fig. 8.

FIGS. 6 TO 8. METHODS OF INSULATING FLOOR CONNECTION TO WALL.

are designed so that the building is divided into several sections and the same temperature maintained on all floors in any one section. Where this idea is followed the individual floors are not insulated. Unless, however, some proposition similar to this be adopted it is essential that floors be insulated.

The insulating of the basement floor or ground floor is subject to a wide difference of opinion. Some very good authorities claim that unless the room of which the ground floor is the base is to be used as a freezer it is unnecessary to insulate the ground floor other than by a bed of cinders and the concrete floor laid on top. As the earth remains practically constant in temperature at about 55° F. and is a fairly good conductor as well, it would seem that there would be a constant loss unless the floor is insulated with the same care as the walls. It seems advisable therefore to construct a base floor of 3 ins. of concrete placed directly on the ground or on a bed of cinders. On top of this lay a waterproofing coat and then apply the insulation either lith, mineral wool, or sheet cork from 2 to 6 ins., depending on requirements. The insulation should be given a mop coat of waterproofing material and covered with 5 ins. of concrete with 1 in. of finish. It would be possible to use a thinner concrete slab by reinforcing it with a small amount of steel. Fig. 9 shows this construction. In most instances it would be well to reinforce the ground floor as the movement of heavy hand trucks or the dropping of barrels from off of one tier to the floor would tend to crack a plain concrete floor with resulting loss of protection.

Self-supporting floors where insulation is necessary are treated in much the same manner as the ground floor. On the reinforced-concrete or hollow tile bearing slab is placed from 2 to 6 ins. of lith, mineral wool or sheet cork. (See Fig. 10.) This should be laid in pitch or some other waterproofing compound to protect it. On top of the insulation is placed the 3-in. concrete floor having 1-in. finish. This top slab is reinforced lightly to prevent cracking of any character. Where the insulation consists of wood and paper the construction is similar to the walls, only the top concrete is given a wearing surface.

Where the insulated room comes next to the roof, the practice has been of two general kinds: one using an air chamber between the roof and the ceiling, of the top story, and the other insulating the roof itself.

The former method is varied in its applications. In some installations an attic 3 or 4 ft. high has been built and in others a hung ceiling has been specified. With either type the ceiling is insulated with any one of the systems described for protecting walls; that is, sawdust, ground cork or mineral wool can be laid on the attic or air chamber floor (which is the ceiling of the top story room) and protected with a waterproofing coat. Lith or cork boards are usually placed in pitch or asphalt and covered with asphalt paper. Fig. 11 shows this type of roof protection—the detail calling for a specially built in attic floor.

The insulating of the roof can be handled by placing the insulation directly on the roof construction. In this case the insulation should be preferably of sheet cork or lith laid in a waterproofing substance. On top of the sheet cork is then placed from 1 to 2 ins. of concrete as a protection and then the regular roof waterproofing is applied. The only serious objection to laying the insulation directly on the roof is that in event of a leak in the roofing developing the percolation of water would spread through the insulating material and show on the ceiling at some point possibly quite a distance from the original leak. This would require ripping up a considerable area of the insulation and roofing in order to locate the break, whereas in the hung ceiling method the roof leak could be detected accurately and at much less expense than in the former case.

Where the temperatures maintained in the rooms in successive stories are not the same it is necessary to insulate the columns, otherwise loss will occur rapidly. This is done in a manner similar to the insulation of the walls. The only special point to be watched is the joining of the floor and column insulation so that it is made continuous.

When buildings are divided into separate fire risks by fire walls the conduction from one floor risk to the other through the continuous floor can best be avoided by carrying each section or risk on its own columns instead of using a common column between the two. This adds very little to the cost of the building, allows a continuous fill of insulating material between each fire risk, and not only prevents conduction through the columns, but also prevents conduction through the floor by reason of the insulation passing by the ends of the beams. This same idea applies favorably to the detailing of elevator shafts and stair walls. If the framing of these shaftways is exposed sweating or frosting may be expected from each floor. Sufficient clearance should be provided therefore to allow of the carrying up of a separate tile or concrete wall inside the shaft and tied to the skeleton of the building as few times as consistent with safety. Between this inner partition and the shaft lining itself a continuous band of insulation should be provided.

The doors and windows in cold storage plants are a serious problem. Of the latter there are generally very few, if any. Of doors, however, there are quite a number always necessary. Wherever any openings are required in the partitions or in the fire walls or from the receiving rooms to the refrigerating chambers double fireproof doors have to be installed. The underwriters have approved making one of these doors a swing refrigerator door covered with galvanized iron or tin in accordance with the standard fire door specifications, and the other door an ordinary fireproof door held open with a fusible link. The best types of refrigerator doors have two seals of contact between the door and the frame. Self-tightening fasteners and adjustable spring hinges keep the door tight making an air tight opening which tends to prevent the spread of fire or the loss of refrigeration through the joints. The doors themselves are hollow, the sides being of wood metal covered, with an insulating material filling the air space. The cold storage building windows are designed along the same lines as the doors. The glazing consists of as many thicknesses as conditions require. All the glass is nested in one sash which is hung with adjustable spring hinges and held closed by a special fastener. The outside thickness of glass is generally reinforced with wire. The double seals of contact make an efficient joint.

One of the main topics of discussion on this subject of insulation of cold storage buildings is the question of fireproofness and the conditions which attract the interest of the underwriters and influence their verdict as expressed in the rates they are willing to quote. On cold storage buildings with solid insulation fireproofed skeleton, and concrete or brick walls the fire risk is at a minimum. As noted at the beginning of this paper the insurance companies have been obliged to be very severe on the ordinary cold storage proposition because in the early day with wood walls and filled air spaces ideal fire conditions were established. In the more modern buildings even it was felt that the insulation increased the fire hazard owing possibly to the opportunity offered a fire to smolder in the walls. But with best up-to-date practice the insulation is really a help in making the properly designed cold-storage building fireproof. There are instances on record where cork and similar insulating materials have withstood fires in a manner to compare favorably with ... insulation from the very reason for its ... vents heat from reaching the walls. In ... has been found that so little heat passes ... insulation to the walls that they were ... other cases slow burning constructions ... resistant insulation have confined to single ... which in the usual type of building would ... from cellar to roof and gutted the structure.

The questions of the relative efficiency ... ent insulating materials are too complex ... tion which can be obtained is too partisan ... character to permit of much being said ... limits of this paper. There is praiseworthy ... work being done at several of the great technical schools and one company engaged in the manufacture of an insulating material maintains in Pittsburg an experimental station which is carrying on investigations of value.

Now as to which building material is the best suited to structures intended for cold storage purposes. The majority of plants up to the present time have been built of brick or stone with structural steel frames and terra cotta or brick floors. There are, however, a considerable number of artificial refrigeration establishments in which reinforced concrete has played an important structural part. These are the more modern ones. Mr. John E. Starr of New York City states that another point may be mentioned which shows the great stability of reinforced concrete construction for beams, girders, . . . floors. In the house . . . at St. Paul the construction of the concrete work was commenced in the fall and continued through the winter. . . . The refrigeration was turned on in hot weather and the first floors refrigerated were in the center of the building at very low temperatures, ranging from zero to 5° F. below zero and up to 35° above. It will readily be seen that under these conditions the building was subjected to the most severe strains that could be conceived of in the way of expansion and contraction and yet there was not a sign of a flaw or a crack in any column, girder, beam, or floor, and this on a comparatively green building, which of course is growing stronger every day.

When a statement such as this is made the availability and the advantages of reinforced concrete for such construction may be considered as well established.

In addition to the ability of concrete to withstand the wracking which putting a building into a state of refrigeration sets up, its fireproof and waterproof qualities must be admitted; and the elimination of vibration in a reinforced-concrete mill even under the most severe machinery movements must have some bearing on the possibility of this material being best for cold storage plants when vibration means unsettling insulation, cracking waterproofing and doing similar other damage. Taking into account also the load sustaining properties of concrete which it is almost unnecessary to mention to those who know the substance at all, and regarding with a critical eye, its permanency, which means elimination of repairs and maintenance charges, the unbeliever must acknowledge that reinforced concrete certainly is entitled to trial for cold storage structures.

That it has been used in face of the sharp competition it has had to meet is due to the readiness with which it meets all specifications and the economy it effects when compared to other fireproof structures. Reinforced-concrete buildings of reasonable size that are costing $60,000 and upwards can be and actually have been erected at costs running from 10% to 20% lower than the best bid received for steel construction on the same plans. On medium propositions concrete will not exceed first-class mill construction by more than 10% and in several instances where the estimated cost of the building has run over $150,000 the figures submitted by the concrete contractors have been slightly lower than those of the mill construction men.

For special construction necessitating long spans, irregular column spacing, unusually heavy loads, curtain walls, or like unique features, concrete is particularly well adapted and under just such conditions has proved itself time and time again.

The insurance question which is so much mooted among the cold storage interests has direct bearing on the use of reinforced concrete. Baltimore and San Francisco tested concrete and found it good. Individual fires like

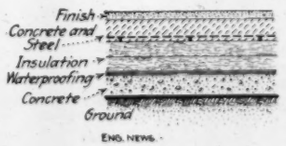

Fig. 9. Insulation of Basement.

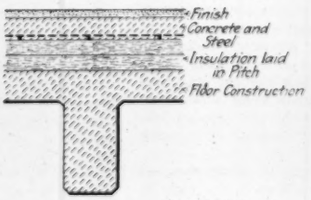

Fig. 10. Floor Insulation.

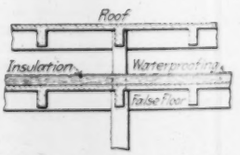

Fig. 11. Roof Insulation.

the Pacific Coast Borax Company Building in Bayonne, N. J., the [illegible] Company factory in New York City, and the [illegible] Motor Car Works at Dayton, O., showed the materials under most trying fire conditions and demonstrated [illegible] keen sighted insurance men as typified by [illegible] tual Companies are conservative when they offer [illegible] the lowest rates on concrete buildings that have [illegible] been written on similar structures, and still more [illegible] when they urge the adoption of reinforced concrete [illegible] industrial buildings.

The writer of this paper extends his acknowledgments for valuable assistance in preparing the same to the following [illegible] Mr. H. E. Plumer, C. E., Contracting Engineer, [illegible] per Construction Company; Mr. Junius H. Stone, Manager of the Nonpareil Cork Works; Mr. John E. Starr, Assoc. Am. Soc. Refrigerating Engineers; Mr. Van Rensselaer H. Greene, Chief Engineer of the Manhattan Cold Storage Plant; Jones Cold Storage Door Co.; U. S. Mineral Wool Co.; Union Fiber Co.; and to a number of prominent reinforced-concrete construction companies. He would thank them for their courtesy in allowing him the use of their literature, drawings and photographs.

The Report of the Board of Engineers on the Panama Canal Work.

To the Senate and House of Representatives:

I submit herewith the report of the engineers appointed by me to accompany the ex-Secretary of War, the Honorable William H. Taft, to the Isthmian Canal to look into the condition of the canal work, and especially to report upon the feasibility and safety of the Gatun dam project with a view to deciding whether or not there should be any change in the plans in accordance with which the canal is being constructed, these plans having been adopted by the Congress.

I am happy to report to you that the accompanying document shows in clearest fashion that the Congress was wise in the position it took, and that it would be an inexcusable folly to change from the proposed lock canal to a sea-level canal. In fact, this report not only determines definitely the type of canal, but makes it evident that hereafter attack on this type—the lock type—is in reality merely attack upon the policy of building any canal at all.

The board of engineers who signed this report are of all the men in their profession, within or without the United States, the men who are, on the whole, best qualified to pass upon these very questions which they examined. I commend to you the most careful consideration of their report. They show that the only criticism that can be made of the work on the Isthmus is that there has sometimes been almost an excess of caution in providing against possible trouble. As to the Gatun dam itself, they show that not only is the dam safe, but that on the whole the plan already adopted would make it needlessly high and strong, and accordingly they recommend that the height be reduced by 20 ft., which change in the plans I have accordingly directed.

Every American citizen should feel not merely gratification but a very keen sense of pride in the statement made by this distinguished body of engineers as to the way in which the work has been done and in which it is now proceeding. The American people are to be heartily congratulated on everything of importance that has been done in connection with the building of the Panama Canal.

(Signed) THEODORE ROOSEVELT.

The White House, Feb. 17, 1909.

To the President:

Sir: In accordance with your instructions we have visited the Isthmian canal in company with Hon. Wm. H. Taft, and have examined the work in progress and the plans for the structures as far as now developed.

We have given especial consideration, under the instructions of Mr. Taft, to the foundations for the Gatun dam, and the feasibility of constructing and maintaining thereon a safe dam for retaining water at 85 ft. above sea level.

We have examined the slides in the banks of the canal, and the surveys, plottings and sections that have been made of them. The subsidence in the fills in the toes of the dam and in the railway embankments has also been examined; and we have considered the effect of the qualities of materials thus disclosed upon the construction of the various works and upon their ultimate stability.

We have also considered the evidence that has been accumulated as to the permeability of the different materials and the possible loss of water by percolation through the bed and banks of the future Gatun lake; and the question whether such loss of water by seepage would result in materially reducing the water supply or in undermining and ultimately crippling the structure.

Gatun Dam.

The Gatun earth dam is the central point of discussion, and we were instructed by Mr. Taft to give it first consideration in the light of all new evidence.

We are satisfied, both from the records of the experiments that have been made, and from our own personal examination of the materials, as seen in cuts now open, and as disclosed by samples from test borings, that there will be no dangerous or objectionable seepage through the materials under the base of the dam; nor are they so soft as to be liable to be pushed aside by the weight of the proposed dam so as to cause dangerous settlement.

We are also satisfied that the materials available and which it is proposed to use are suitable and can be readily placed to form a tight, stable and permanent dam.

The type of dam now under construction is one which meets with our unanimous approval. It is a combination of rock-fill and hydraulic fill, in which the exterior faces are to be composed largely of rock of all sizes obtained from the canal excavation, dumped and laid on slopes much flatter than are ordinarily found in earth dams, while the interior of the great mass will consist of clayey material obtained by hydraulic dredging from large deposits at a little distance from the dam, and carried by water through pipes to the places where it is to be used. The material as delivered is a mixture of earth and water. The material held in suspension slowly deposits, finally forming a solid, water-tight embankment. The pond necessarily maintained on the top of the dam during construction tests the embankment at all stages of its growth, searches out any weak points and leads to the closure of any voids or cracks.

The most practical question in the construction of the Gatun dam is the possible slipping and sliding of the materials underneath and in the body of the dam. These materials, speaking broadly, are of a clayey nature, generally impervious to water, but sometimes slipping when subjected to heavy unbalanced pressure or on high, steep slopes when saturated with water. In this respect the materials differ radically from the sandy and gravelly materials which have been frequently used in the construction of other earth dams.

In order to build a dam of these clayey materials that will be stable and permanent, it is necessary that the slopes should be flatter than would be needed to secure the stability of a dam of silicious, sandy or gravelly materials.

The evidence that has been accumulated as to the degrees of slope that are stable with these materials seems to us conclusive. The fact that the materials are slippery does not mean that a dam built from them is necessarily less stable than a dam built of materials that do not slip so easily. It does mean that, in order to secure stability and permanency, the dam must be built with a greater thickness at the bottom. The dam as proposed is more than a third of a mile in horizontal thickness at its base, including the rock-fill portions.

The design upon which the work is now being prosecuted abundantly fulfills the required degree of stability, and goes far beyond the limits of what would be regarded as sufficient and safe in any less important structure.

As a matter of convenience and economy during construction, materials have been piled up on slopes much steeper than those contemplated in the finished work. Generally, the materials so placed have remained in position; but in some cases slips have occurred. The occurrence of these slips is of no serious consequence, either in the practical execution of the work, or in the ultimate stability of the structures. We can readily understand how incorrect deductions may have been drawn from these occurrences, especially by those not fully informed as to the character of the materials, and the ample dimensions and much less steep slopes of the proposed structures in their final form.

We were requested to consider the proper height for the crest of the Gatun dam, and after consideration concluded that it could be safely reduced 20 ft. from that originally proposed; namely, to an elevation of 115 ft. above sea level, or 30 ft. above the normal level of the water against the dam. We are also of the opinion that the sheet piling recently proposed under the base of the dam may be safely omitted. The narrow cut-off trench now in progress through the upper earth stratum on Gatun island and elsewhere and designed to be refilled with sluiced material should be continued.

Changes in these respects will facilitate the work of construction, and will reduce somewhat the cost of the proposed work.

A full study of all the data at hand and of the materials, and of the plans that are proposed with the above modifications leaves no doubt in our minds as to the safe, tight and durable character of the Gatun dam.

Changes in Plan of Canal.

It was suggested to us by Mr. Taft that we give special consideration to those changes which have been made in the plans of the minority of the Board of Consulting Engineers of 1905 since the adoption of the project.

Change in Position of Lower Pacific Locks.—One of the most important of these changes is the moving of the lower locks on the Pacific end of the canal from La Boca, on the shore of Panama Bay to Miraflores, about four miles inland.

This change involved the abandonment of the construction of two earth dams at and near La Boca, and the substitution of about four miles of deep sea-level channel 500 ft. wide from La Boca to Miraflores in place of a wider channel through the lake that would have been created by the dams.

Before this change was made, work had been commenced upon the toes of one of the dams. The material had been piled up to a considerable height on slopes steeper than were capable of being supported by the underlying material. Under these conditions, settlements occurred with lateral displacement of some of the underlying material. Your Board, after carefully inspecting the ground and the partially completed work, is of the opinion that these settlements cause no reason to doubt the stability of the proposed dams. We are unanimously of the opinion that stable and water-tight dams of substantially the proposed dimensions could have been constructed on the proposed sites without recourse to dredging out the underlying soft material.

The report of the minority of the Board of Consulting Engineers of 1905 recognized that an objection might be made from a military point of view to placing locks on the shore of the bay, exposed to guns of hostile ships. We now understand that the controlling reason for the change was a military one. This change in the plans will result in an increase in cost of the canal by an amount judged from evidence at our disposal to be not less than $10,000,000. We are informed, however, that this change would greatly lessen the cost of fortification.

Increased Width of Canal.—Another change is the increase of the minimum bottom width of the canal from 200 ft. to 300 ft. This applies to a length of about 4.7 miles in the Culebra cut. We understand that this change will increase the cost of the work by about $13,000,000. The work upon the excavation of the Culebra cut under the revised plan has now so far advanced that this widening will not delay the completion of the canal.

The widening will permit ships to pass one another in this portion of the canal, as they may under the original plan in all other portions, and will otherwise facilitate navigation through it.

If slides occur after the completion of the canal, the wider canal is not as likely to be blocked as a narrower one.

We understand that this change was authorized directly by you on the presentation of its advantages by the Chief Engineer, and we merely call attention to it as one reason for the increased cost of the canal.

Increased Size of Locks.—Another change is the increase of the dimensions of the locks from 95 × 900 ft. to 110 × 1,000 ft. The increase in width we understand has been made in compliance with a request from the General Board of the Navy Department, in order to allow the passage of the largest war vessels contemplated. A large increase in cost is involved in these enlarged dimensions.

Changes in Breakwaters.—An important change is proposed in the location of the breakwater at the Atlantic end of the canal. The plan provisionally adopted by the Board of Consulting Engineers of 1905, and adopted for the purpose of estimate by the minority of that board, was for a breakwater generally parallel with the channel, which included less than one-third of Limon Bay; whereas the breakwater in the location now proposed will protect the entire bay and furnish a more commodious harbor, not only for ships using the canal, but for all other shipping which makes use of the port. A considerable increase in cost is involved in this change.

We had an opportunity to view the present harbor during what is said to have been the only severe "Norther" of the past two years, and have no doubt that a good breakwater is a desirable adjunct to the canal. We are not prepared to pass on the precise location, form, or cost of this.

A change of less importance has been made at the Pacific end by relocating the dredged channel leading to deep water, and increasing its width from 300 ft. to 500 ft.; and by constructing a breakwater from the shore at La Boca to Naos Island with material excavated from the Culebra cut. This breakwater, now under construction, serves to prevent currents across the canal cut, and tends to prevent deposits in the dredged channel and to increase the safety of navigation. The breakwater may also serve to carry a roadway to Naos Island. These changes involve some additional expense.

Relocation of Panama Railroad.—The alinement of the Panama Railroad has been materially changed south of Gatun. This change was made because it was found that the swamp near the Gatuncillo River would not support the very high railroad embankment required, if made with ordinary slopes, and a line crossing at a point higher up the river was selected, which does not, however, materially increase the length of the railroad. The construction of the railroad will cost much more than was estimated by the minority of the Board of Consulting Engineers, who were unable to procure surveys of the proposed location. The recent change in location affords more ample and convenient anchorage immediately above the locks.

Other Changes.—Some further changes or additions which have not yet been fully worked out have been mentioned to us as likely to be made as the work progresses; namely, the dredging out of a broad anchorage basin immediately downstream from the Gatun locks, another for anchorage and room for turning of long ships near La Boca, and possibly another just below the Miraflores locks. These can all be delayed until the completion of the main work of canal excavation and lock-building, and then executed by the dredges that have done the main work. The work can thus be done without additional equipment and at a low price per cubic yard.

Present Condition of Work.

It has been suggested that we report upon the condition of the work and the progress being made, and, if found possible in the time at our disposal, upon the probable time of completion.

Organization.—We have seen the work under way on all parts of the canal. We have become acquainted with the engineers in responsible positions and have noted the organization and equipment. It is our impression that the work is well organized and is being conducted energetically and well.

The work is done by day labor and not by the contract system. The men are well paid, well housed, well fed and well cared for in case of sickness or accident. Houses, furniture, fuel, water, drainage and lights are furnished to employees without cost. Roads are built, schools supported and Young Men's Christian Association buildings provided, which are practically club buildings. Parts of the running expenses are also paid. The premises are cleared and drained and the grass kept cut. The climate is especially adapted to outdoor life, and the ample porches, entirely enclosed by bronze wire screens, give the greatest facility for this. We are especially pleased with the architectural arrangements of the houses. They are admirably adapted to the climatic conditions.

Bachelor quarters and hotels furnishing meals at moderate prices are also provided by the Government. Hospitals are provided, free medical attendance is furnished to employees, and medical attendance at low rates is supplied to families of employees.

A limited amount of free transportation, namely, one excursion trip each month to any station, is furnished on the Panama Railroad to employees, and half rates are given in all other cases, and also half rates to families of employees. Free transportation in some cases, and in all other cases transportation at reduced rates to and from the Isthmus is provided to employees and their families.

Six weeks leave of absence each year, with full pay, is given to all monthly employees, and this includes not only office and engineering forces, but also the mechanical forces on the monthly basis.

The medical and sanitary department is especially to be commented for its success in exterminating yellow fever and controlling malaria, and for other measures which have made the Isthmus a thoroughly healthful place in which to live.

The cost of the sanitary department, which represents the cost of keeping the Isthmus healthful, amounts to about two million dollars per year. This is a large sum, but the work is well done, and any decrease in the efficiency of the sanitary service might readily prove disastrous to the prosecution of the main work.

We believe that in no other great construction work has so much been done for employees in the way of furnishing necessities, comforts and luxuries of life at the cost of the work, as has been done in this case. This is one reason for the high cost of the canal.

Progress and Time of Completion.—We have examined diagrams and statistics showing the amount of work accomplished by years and by months since the work was taken over by the United States, and showing the amounts of the various classes of work remaining to be done and the estimated rates of progress and times required for completion. It has been impossible for us to check these in detail, but we have compared them with other estimates, and with the work obviously done, and they seem reasonable to us. In the light of this showing we see no reason why the canal should not be completed, as estimated by the Chief Engineer, by Jan. 1, 1915; in fact, it seems that a somewhat earlier completion is probable if all goes well, but in view of possible contingencies it is not prudent at this time to count on an earlier date.

Cost of Work.—In examining the expenditures thus far made, it must be borne in mind that large sums have been paid for steamships, dredges, steam shovels, locomotives, cars, tracks, shops, and all the equipment that is necessary to prosecute a work of this magnitude, and also that large sums have been spent for dwellings, offices, buildings of various kinds, for waterworks, sewers, paving, and other equipment, and that these expenditures have been made, in large measure, for the whole work, and that corresponding disbursements hereafter will be very much less in proportion than they have been to date.

Col. Goethals has presented to us estimates of the quantities of materials, and cost, involved in the construction of the canal as now planned, including all disbursements thus far made and the estimated amounts required for completion. These cover the greater excavation, the increased size of locks, the extra canal channel required by moving the Pacific locks from La Boca to Miraflores, improved harbor arrangements at Colon, and all other changes which have been adopted or are now seriously contemplated. The payments to the New Panama Canal Co. are included, and also the payments to the Republic of Panama, and the cost of sanitation and zone government, for which items the Board of Consulting Engineers of 1905 stated that it presented no estimates.

The estimates and allowances seem ample to us. In some items it would seem that considerable reductions could be made; on the other hand, the work is large and unforeseen contingencies must be expected, so that it may be that the aggregate estimate as presented is not too large.

After deducting $15,000,000, representing the estimated receipts from the return of money loaned the Panama Railroad, and from the collection of water rates to cover the cost of municipal improvements made in Panama and Colon, and from miscellaneous sources, this present estimate of the complete cost of a lock canal amounts to $360,000,000.

In making this estimate no reduction has been made for whatever salvage may be realized from the construction plant at the termination of the work, which plant has cost to date about $30,000,000.

The cost of the canal, as estimated in 1905, is frequently stated to be $140,000,000, but this is incorrect, as the minority report expressly excluded sanitation and zone government, and the payments to Panama and the French Company had already been made. Adding these amounts, using the present estimates of sanitation and zone government, we have, in round numbers, the following:

Estimate of the Minority of the Board of Consulting Engineers for the cost of construction, exclusive of sanitation and zone-government	$140,000,000
Payments made to the Republic of Panama and to the New Panama Canal Company	50,000,000
Sanitation and zone government, as now estimated	27,000,000
Total	$217,000,000

The difference between this cost, and the total cost as now estimated is therefore $143,000,000. Of this amount nearly one-half can be accounted for by the changes in the canal and appurtenant works to which we have already referred, and the remainder is to be attributed mainly to the higher unit cost of the different items of the work, caused in part by the higher prices for plant, supplies and labor which have prevailed in the United States since the estimate of 1905 was made, and which made it necessary to offer very high wages and special inducements in order to obtain the requisite force in a locality where the reputation for health was not good in the earlier years, in part to the adoption of an eight-hour day for most of the work instead of a ten-hour day, in part to the much greater expenditure for housing and care of employees and for auxiliary works than was anticipated, and in part, in our opinion, to the evident purpose to make the estimates ample, and to provide liberally for contingencies.

When the work at Panama is completed, in addition to having the canal the United States will own the Panama Railroad and the steamship line operated in connection therewith.

Type of Canal.

In view of the fact that the cost of the lock canal as now proposed will largely overrun the estimate of the minority of the Board of Consulting Engineers of 1905, and that the excavation in the Culebra cut is being made somewhat more rapidly than was anticipated, we have considered in a very general way the relative cost and time of construction of a sea-level canal.

Most of [illegible] factors which have operated to increase t[illegible]st of the lock canal would operate with simil[illegible]ect to increase the cost of the sea-level can[illegible]nd at the present time there are additiona[illegible]ors of even greater importance to be consi[illegible]as affecting the time of completion and cost [illegible] the sea-level canal. One of these is to be foun[illegible] the Gamboa dam, proposed to be nearly 2[illegible]t. in height above its foundations, which w[illegible] be about 60 ft. below the normal river lev[illegible] Prior to the construction of this dam a long a[illegible]eep diversion channel must be provided of [illegible] greater magnitude than that for the Gatun da[illegible] which has been about two years in progress[illegible]d is not yet completed.

Judging [illegible]y the time required for the construction of dams of similar magnitude in the United States, it is probable that were work on the Gamboa dam to be started as soon as possible, this one feature of the sea-level project of the Board of Consulting Engineers of 1905 could not be completed until after the time required for the completion of the lock canal. The construction of this dam at Gamboa for the control of the Chagres is an essential preliminary to the excavation of the sea-level canal for the thirteen miles from Bohio to Bas Obispo. Furthermore, in addition to the Gamboa dam, the sea-level project provides for building for the control of tributary streams three large dams, the sites of which have not been examined.

Work is already far advanced on nearly all parts of the lock canal, and a change in the type would result in abandoning work done which represents large expenditure.

Under the plan now being carried out, the River Chagres and each of the other rivers on the Isthmus tributary thereto is made an ally of the project. The waters of these rivers are handled economically and in such a way as to facilitate the operation of the canal. With the sea-level project, these rivers instead of being allies would be enemies of the canal, and floods in them would greatly interfere with the work.

The excavation of the canal would be carried to 40 ft., or more below sea level and to a much greater depth below the bottoms of the valley in which the upper streams now flow.

It would further be necessary to cut long and large diversion channels on each side of the canal, for streams entering the Chagres Valley. The cost of such lateral channels to protect the Culebra cut alone from the comparatively small streams formerly entering it, including work done by the French, has probably been not less than $2,000,000. The channels required for the lower valley of the Chagres would be necessarily much longer, larger and far more expensive.

Rock Excavation Under Water.—Much has been said about the economy of excavating rock under water by modern appliances as compared with the cost of such excavation in the dry with steam shovels after blasting.

We concur in the opinion of those in charge of work at the Isthmus that it is more economical, where the conditions are favorable, to excavate rock in the dry than any under-water process now in use. Experience is not yet available to us which will justify the belief that, with the depth of cut and the quality of rock found on the Isthmus, the general adoption of subaqueous methods would prove more expeditious or cheaper.

It is probable that more economical subaqueous methods will be sometime developed, but it would not be wise to base a change in plan of important work upon prospective results to be obtained by any method not yet thoroughly tried.

Earthquakes.—It has been suggested that the canal region is liable to earthquake shocks and that a sea-level canal would be less subject to injury by earthquakes than a lock canal.

We have seen, in the City of Panama, the ruins of an old church, said to have been destroyed by fire, containing a long and extremely flat arch of great age, which convinces us that there has been no earthquake shock on the Isthmus during the 150 years, more or less, that this structure has been in existence, that would have injured the work proposed.

Dams and locks are structures of great stability and little subject to damage by earthquake shocks. The successful resistance of dams and reservoirs supplying San Francisco with water, even when those structures were located near the line of fault of the earthquake, gives confidence in the ability of well designed masonry structures and earth embankments to resist earthquake shocks.

We do not regard such shocks as a source of serious damage to any type of canal at the Isthmus, but if they were so, their effect upon the dams, locks and regulating works proposed for the sea-level canal would be much the same as upon similar structures of the lock canal. The Gamboa dam for controlling the floods of the Chagres in connection with the sea-level canal provides for a lake having an area of 29 square miles when full, and if this water were suddenly let loose into the sea-level canal it would seriously injure large portions thereof and wreck ships therein. A similar result would be reached if the other three dams of the sea-level canal retaining lakes, having an aggregate area of ten square miles, were to be suddenly destroyed.

Water Supply.—We believe that the sufficiency of the water supply for a lock canal has never been seriously questioned. It is true that during the dry season the natural flow of the streams would not be sufficient to furnish the water required for numerous lockages. There would even be times when the natural flow would not suffice to make good the loss by evaporation from the surface of the water in Gatun lake. During the rainy season there is a great excess of water which can be readily stored in Gatun Lake, with its area of 163 square miles. It is proposed to fill this lake during the rainy season 2 ft. above its normal level, and to draw it as needed during the dry season. It is computed that by drawing it 5 ft. below normal level, which draft would leave 40 ft. of water through Culebra cut, the supply in a dry year would be sufficient to serve from 30 to 40 lockages up, and an equal number of lockages down, daily. Each lockage might consist of a single large vessel, or a fleet of smaller vessels, capable of being in the lock at one time, as is common at Sault Ste. Marie. For comparison the published record shows that an average of only twelve ships per day passed through the Suez Canal in 1907.

Ultimately, if needed for increased traffic, additional water may be held from wet seasons and made available in dry ones. This may be accomplished either by raising further the high-water level in Gatun Lake, or by lowering the low-water level in the lake, this lowering being accompanied if necessary by the deepening of the canal, or storage may be provided by an entirely independent reservoir, for which there are excellent sites.

From our examinations in the neighborhood of Gatun dam, we can find no reason to apprehend important loss of water by seepage through the ridges surrounding the lake, while in our judgment the bed of the lake will be practically impervious to water.

The water supply in sight is so much greater than any need that can be reasonably anticipated that the best method of securing more water when the time of need arrives does not require to be considered now.

Conclusions.

Your Board is satisfied that the dams and locks, the lock gates, and all other engineering structures involved in the lock canal project are feasible and safe, and that they can be depended upon to perform with certainty their respective functions.

We do not find any occasion for changing the type of canal that has been adopted.

A change to a sea-level plan at the present time would add greatly to the cost and time of construction, without compensating advantages, either in capacity of canal or safety of navigation, and hence would be a public misfortune.

We do find in the detailed designs that have been adopted, or that are under consideration, some matters where other arrangements than those now considered seem worthy of study. As these proposed changes are of a tentative nature and do not in any case affect the main questions herein discussed, they are not taken up in this report.

Very respectfully,

Frederic P. Stearns,
James D. Schuyler,
Arthur P. Davis,
Isham Randolph,
Henry D. Allen,
John R. Freeman,
Allen Hazen.

Washington, D. C., Feb. 16, 1909.

The Records of the Engineers Forming the Advisory Board on the Panama Canal.

In some of the newspaper criticisms of the report made by the engineers who accompanied Mr. Taft to the Isthmus, attacks have been made upon the professional standing of these gentlemen and on the reliability of their report. It is perhaps almost needless to record in these columns the professional experience and standing of these well-known engineers. Yet as there may be some of our readers who may not be familiar with all of these gentlemen, we have thought it worth while to give some of the main facts regarding their professional record.

Mr. F. P. Stearns ranks without question among the most eminent hydraulic engineers in the United States. He was for some 17 years in charge of engineering work for the city of Boston, holding positions of great responsibility. In 1886 he became Chief Engineer of the Massachusetts State Board of Health, and was in charge of the important engineering work carried on by that body for nine years. In 1895 he became Chief Engineer of the Metropolitan Water Board and its successor, the Metropolitan Water and Sewerage Board; and the works for the water-supply and sewerage of the Metropolitan District of Boston, which rank among the greatest in this country, and, indeed, in the world, were carried out under his direction. During the past decade he has had a consulting engineering practice extending from one side of the continent to the other, and has had connection with the most important engineering works in the country. He has been honored by his fellow-members in the engineering profession with the presidency of various professional societies, including the American Society of Civil Engineers.

Mr. James D. Schuyler, of Los Angeles, Cal., is one of the best known engineers in the West, and has had a particularly large and important experience in the construction of dams. His professional work dates back to 1869. In 1887-88 he designed and built the famous Sweetwater Dam in California, and later the Hemet Dam, the highest masonry structure in California. He has built or supervised important dams and reservoirs in connection with the water-works of Denver, Colo., Portland, Ore., and Columbus, O. He has been three times called to Hawaii to report on irrigation works, and has supervised the construction of dams in California, Idaho, New Mexico, Colorado, Mexico, Brazil and British Columbia. He is building several monster dams in Mexico at the present time, and is in charge also of important works in British Columbia. He is the author of a treatise on dam construction which is a standard authority among engineers.

Mr. Arthur P. Davis has been for the past four years Chief Engineer of the U. S. Reclamation Service, in charge of all the engineering work in the Service, involving the expenditure to date of about $50,000,000 and an ultimate expenditure on the works now started of about twice this amount. For four years previously he held the position of Assistant Chief Engineer. Mr. Davis has had previous experience with the canal enterprise. He was a member of the engineering staff of the Nicaragua Canal Commission of 1897-98, and was Chief Hydrographer for the Isthmian Canal Commission of 1900-01. His experience in the Reclamation Service and in the construction of dams under a great variety of conditions of climate and soil, as well as his previous experience on the Isthmus, particularly

fitted Mr. Davis to give authoritative opinions on the questions submitted in connection with the recent visit to the Isthmus of Mr. Taft.

Mr. Isham Randolph has had an engineering career extending over more than 40 years. Since 1893 he has been in full charge of the work of the Sanitary District of Chicago, and the Chicago main drainage canal, which ranks among the greatest canals of the world, is the best monument to his engineering ability. Besides this work, Mr. Randolph has been consulted in connection with many important enterprises.

Mr. John R. Freeman is an unusual combination of the business man engaged in the direction of important enterprises and the engineer of high professional standing. Mr. Freeman was among the early graduates in civil engineering at the Massachusetts Institute of Technology, and for ten years thereafter was engaged with the Essex Co., of Lawrence, Mass., in connection with the hydraulic work of that great water-power-development company. In connection with this work in hydraulics, Mr. Freeman had occasion to make special investigations of the fire protection of mills, and has done much to develop and put on a professional basis the comparatively new branch of fire-protection engineering. From engineering work for insurance companies making a specialty of the protection of New England manufacturing plants, Mr. Freeman was rapidly promoted to high executive position, and he is now the president of insurance companies charged with the protection of property valued at over $350,000,000. Coincident with this, Mr. Freeman has carried on a wide practice as a consulting engineer, and has been called upon for advice in connection with many of the largest and most important problems in hydraulic work located in all parts of the United States. He has served as President of the American Society of Mechanical Engineers, as President of the Boston Society of Civil Engineers, and as Vice-President of the American Society of Civil Engineers. Few men in the whole profession have higher ideals of engineering and of the responsibilities of an engineer to his fellows and to the public, than Mr. Freeman.

Mr. Allen Hazen is the senior member of the engineering firm of Hazen & Whipple, of New York City, and has had some 20 years experience in engineering work since his graduation from the Massachusetts Institute of Technology. While Mr. Hazen's early work was largely in the field of water filtration, he has in recent years become well known in connection with general hydraulic engineering enterprises and his consulting work has extended as far away as Australia, where he was summoned a year ago for advice in connection with large works for the water-supply of Brisbane.

EITHER THE TUBERCULIN TEST OR PASTEURIzation, in connection with the milk supply of Chicago, was made compulsory by a city ordinance that went into effect on Jan. 1, 1909. The ordinance provides that beginning with that date no milk nor milk products must be supplied in Chicago unless the milk is either produced from cows which have been shown by the tuberculin test to be free from tuberculosis, or unless the milk has been pasteurized. If pasteurization is chosen, rules and regulations formulated by the Department of Health must be followed. The ordinance does not fix the temperature at which pasteurization must be carried on. Instead, the type of pasteurizer proposed for use must be submitted for test at the laboratory of the Health Department, in order, to quote from the bulletin of the Department for Jan. 2, 1909,

> to determine the temperature at which tubercle bacilli and other pathogenic germs and 99% of all bacteria are killed. The temperature at which this is accomplished is fixed for that particular pasteurizer and the machine must be run at that temperature and a record kept by means of a recording thermometer.

The bulletin for Jan. 2 states that large numbers of tuberculin-test certificates are being received daily, and that within a few days it is hoped that 8,000 to 10,000 certificates will be on file. From this it was concluded that there would be a plentiful supply of milk from tuberculin-tested cows, so that milk distributors not provided with pasteurizers would have no difficulty in securing milk for their customers. The pasteurizing plants, it may be inferred, must be located in the city.

Preliminary Recommendations of the Engineering Committee of the Pittsburg Flood Commission.

Destructive floods at Pittsburg in the winters of 1907 and 1908 led to the appointment of a Flood Commission under the auspices of the Pittsburg Chamber of Commerce. This Commission, of which Mr. Geo. M. Lehman is secretary, has assigned its work to various committees, among which is an Engineering Committee. This Committee has recently submitted a progress and program report, as follows:

> The Flood Commission, which was appointed by the Chamber of Commerce, of Pittsburg, for ascertaining remedial measures necessary for the avoidance of the enormous damage and suffering from floods, so frequently occurring to this city, presents to the public the mode of procedure for the investigation, as recommended by the Engineering Committee of the Commission.
>
> The Engineering Committee, at the request of the Flood Commission, have obtained through the courtesy of the Secretary of State, Washington, D. C., over two hundred reports from U. S. Consuls in foreign countries, bearing on the subject of flood prevention and flood protection. Many of these are technical reports, made by flood commissions appointed by the various governments, and contain much information as to methods used and proposed in those countries. These reports will receive thorough study by the Engineering Committee, to determine whether any of them are applicable to conditions in the Pittsburg district.
>
> Maps have been made, showing the flooded area of the city, which covers about 1,600 acres, and a schedule has also been prepared, from an actual canvass of the affected districts, of the amount of damages caused by three floods, as follows: March 15, 1907, when the height reached was 35.5 ft.; Feb. 16 and March 20, 1908, with heights of 30.7 and 27.3 ft., respectively. In round numbers, the amount of direct damage caused by these three floods was $4,000,000, so far as ascertained, but the total loss, direct and indirect, will vastly exceed this figure, as the Commission has not yet received complete returns from property owners.
>
> After a careful review of the flood conditions in Europe, so clearly outlined in the reports already received, and personal experience with the damages sustained yearly by the ever-increasing floods in Pittsburg, we are convinced that this great subject must be dealt with in its entirety; that it will be impossible to intelligently design a wall around the city without all the facts; as it is necessary to completely and at all times keep out the flood waters, and at the same time design a wall that will be of no greater height than necessary and at the least possible cost to the city.
>
> The Engineering Committee believe it would not be doing its full duty to the Commission, or to the citizens of Pittsburg, if it does not thoroughly consider every possible means for prevention of floods, which are bound to occur from time to time, or the lowering of their heights.
>
> In the study of the problem the Engineering Committee propose to consider the following subjects and their bearing on its solution:
>
> (1) Walls or dikes.
>
> (2) Elevation, or filling in of ground subject to overflow.
>
> (3) Deepening and correction of navigable parts of the river channel.
>
> (4) Storage reservoirs.
>
> (5) Forestation of watersheds and any other means that may seem practicable.
>
> Until a thorough investigation has been made, it is, of course, impossible to determine which of the above plans will give the needed results, or what combination of them may be necessary to accomplish the desired result.
>
> [*Signed*]
>
> E. K. Morse, M. Am. Soc. C. E., Consulting Engineer.
>
> Emil Swensson, Vice-Prest. M. Am. Soc. C. E., Consulting Engineer.
>
> Paul Didier, M. Am. Soc. C. E., Assistant Chief Engineer Baltimore & Ohio R. R., in charge of Pittsburg District.
>
> W. G. Wilkins, Dir. M. Am. Soc. C. E.
>
> A. B. Shepherd, Director Public Works, Pittsburg.
>
> Morris Knowles, M. Am. Soc. C. E., Chief Engineer Bureau of Filtration, Pittsburg.
>
> Julian Kennedy, Consulting Engineer.
>
> S. C. Long, General Manager West Penn R. R.
>
> Geo. M. Lehman, Chief Engineer Erie & Ohio Ship Canal.

[The society membership and other statements have been added by the Editor. Mr. Morse is chairman and Mr. Lehman is secretary of the Committee.—Ed.]

It is interesting to note that the members of the Engineering Committee have been formed into two sub-committees: Flood Prevention, W. G. Wilkins, Chairman; Flood Protection, Emil Swensson, Chairman.

From a statement forwarded to us by Mr. Lehman we take the following figures as to floods at Pittsburg since 1873:

River stage. (feet)	Number of floods—First half	Second half.
22	19	28
26	4	13
30	1	5
35	0	1
Totals	24	47

(The danger line is at 22 ft.)

Editorial comment on the Pittsburg flood studies may be found elsewhere in this issue.

The Use of Metallic Sodium for Drying Transformer Oils.

Methods of adding sodium for the [illegible]d drying of hydrocarbon oils are familiar to [illegible]ists. It is usually thought necessary to di[illegible] off from the sludge to get rid of the caustic[illegible], formed by the reaction between the sod[illegible] and the moisture. That this has not been [illegible]d necessary in working oils on a large sc[illegible]s shown in the following extracts from a [illegible]t on the commercial use of sodium in del[illegible]ating oil for alternating-current transformer[illegible]. This report was made to the Roessler & [illegible]asslacher Chemical Co. and shows the result[illegible] studies by the Niagara Electro-Chemical [illegible] Niagara Falls, N. Y. Metallic sodium is [illegible] of the products of the latter company an[illegible]s marketed through the former.

Recent experience in connection with [illegible] drying of transformer oils has convinced us that b[illegible]he use of metallic sodium, moisture may be compl[illegible] removed from oils which are to be used for insula[illegible] purposes. This means, of course, that the insulati[illegible]qualities of the oil will be raised in a very marked de[illegible].

Sodium has a density of about 0.97 at 150[illegible] (59° F.) and melts at 97° C. (207° F.). In transform[illegible] oil it will sink unless dragged to the surface by hydrogen gas. With water it reacts to form caustic soda and hydrogen. If very much water is present the caustic soda dissolves and in the presence of oil forms a second layer. If very little water is present the caustic soda is formed on the surface of the metallic sodium and may be removed when removing the sodium. When the sodium surface becomes covered with caustic, it is advisable to remelt under oil, not letting the temperature rise above 120° C. (248° F.). After cooling and getting into the shape desired, it is again ready for use for drying more oil. Sodium should always be kept under a good transformer oil.

METHOD OF USE.—One method which we have used with our own transformer oils has been as follows:

The oil on the granulated sodium is poured off and a good transformer oil poured over the metal. To the oil which is to be treated, and which is put into an open tank or barrel, the sodium is added at first very carefully, about one ounce to the barrel. If much hydrogen is evolved, this will be conclusive proof that there is much water in the oil, and the balance of the sodium should be added carefully and in small amounts. The amount which is to be added depends upon the oil, but as a rule one pound to the barrel is usually much more than is required. The oil is then stirred up three or four times a day for a minute at a time. After several days the oil may be removed and tested, but the longer it remains over the sodium the better the oil becomes.

Another method used is to put the sodium in the form of sticks in a cylinder of iron wire of about 28 mesh and hang the cylinder in the oil. This method may be used directly with the static transformer which is in use. The only precautions required being those familiar to all electricians in the avoidance of short circuits.

RESULTS.—We have repeatedly taken oils which broke down at 3,000 volts and by letting them stand over the sodium for two days have brought their breaking points up to 20,000 volts or higher. Results as good as this had been obtained on a large scale by one of the largest power companies.

TO PROTECT THE LOWEST BIDDER and the public interest, the legislature of Indiana has passed a bill providing that when County Commissioners let a contract to another than the lowest bidder, the latter may appeal to the Circuit Court for a review of the award.

A WATER TANK FOR CALCUTTA, which it is said will cost about $2,300,000, has been proposed for some time past. Mr. William H. Michael, U. S. Consul-General at Calcutta, comments on the failure of American firms to compete for the contract (Daily Consular and Trade Reports, Jan. 28, 1908) as follows:

> The contract for erecting the proposed elevated steel reservoir for the additional supply of water to Calcutta will not be awarded until Feb. 4, 1909, and not on that date if the opposition in the council can have its way. The principal bidders are Messrs. Bird & Co., Calcutta, and Messrs. Clayton, of England, whose representative is on hand in person to look after the firm's interest. After all of my trouble to place this big contract before American engineers and contractors and manufacturers of iron and steel, no American firm as a bidder is in evidence. With present facilities for shipping steel in large quantities at a cheap rate of transportation from New York and Baltimore direct to Calcutta, there seems no good reason why Americans should not have tried for this large contract, not only for the money in it, but prestige to be gained by such a contract being successfully performed by an American company.

A brief description of this tank appeared in our issue of March 5, 1908.

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Vermont has joined a number of other States in creating a public service commission having control over the railway, express, telegraph, telephone, gas and electric lighting companies within its borders. While simpler than the legislation of some of the other states, it appears that Vermont has nevertheless given its commission fairly adequate powers to deal with the public service problems which are likely to arise in that state. One exception to this deserves mention. We refer to the following provision:

> Nothing in this act or previous statutes shall be construed as giving the Public Service Commission power to prevent or restrict competition or limit the number of persons or companies who may engage in the business of furnishing light, heat, power, or any other business subject to supervision under the provisions of this act in any town.

It seems strange that in this day of enlightenment any attempt should be made to enforce competition, or, if not to enforce it, to prohibit any effort to limit it. We understand that the prohibition was prompted more particularly by those who wish to see nothing done to injure independent telephone companies. If this, rather than general backwardness in realizing that public service corporations are and should be monopolistic in character, was the motive for inserting the prohibition, then it would seem to have been far better to have limited the prohibition to telephone companies. The day will come, however, when the people of Vermont, as well as of every other state in the Union, will see that poorer service and higher rates are the natural and inevitable consequences of attempts to secure competition in telephone as well as in other public services.

A well-considered program for flood prevention and flood protection studies at Pittsburg has been presented to the Pittsburg Flood Commission of the Chamber of Commerce of that city, by the Engineering Committee of the Commission. The program, which, together with an outline of the work of the committee to date, is presented elsewhere in this issue, includes a proposed study of walls or dikes for protecting Pittsburg against floods, or, presumably as an alternative, the filling of overflowed areas. It is also proposed to study the regulation of the channels of the rivers which give rise to the floods. These are local matters and relate to flood protection. On the flood prevention side, the committee proposes to take up storage reservoirs on the headwaters of the Allegheny and Monongahela, and also the question of reforestation.

The need for flood relief at Pittsburg can be appreciated when it is stated that the estimated damage caused by the floods of 1907 and 1908 was some $4,000,000, not including various indirect damages which cannot be figured. During the last half of the 35-year period beginning with 1873, it was said that there have been 47 floods in which the river levels were above the danger line, against 24 in the first half of the period. It seems very pertinent to inquire why this has been so. Has it been due to contraction of the river channel, to changed conditions in the drainage area, or merely to variations in the amount and distribution of the rainfall? Doubtless the Engineering Committee will answer these questions in due time, if it is afforded full opportunity to carry out its program.

We hope the Engineering Committee will be provided with ample funds, as well as less material but nevertheless very important moral and general public support, for carrying out its proposed investigations. The committee is to be commended for attempting to look upon all phases of the subject, instead of confining its investigations to a single phase—such, for instance, as the possibility of building dikes or walls, or the raising of the level of the overflowed areas. It seems at least quite as important to enter into an investigation of possible means of flood prevention as to confine the studies to flood protection works. Obviously, anything that can be achieved in the way of flood prevention will diminish the necessity for flood protection.

Perhaps it will not be out of place to suggest that the studies proposed by the Engineering Committee might be made still broader, possibly by cooperation with the State Water Supply Commission and the U. S. Geological Survey, in order that the fullest possible conservation and utilization of the streams involved be taken into account. Navigation interests on the Ohio should also be considered.

The names of the members of the Commission are given in the article published elsewhere in this issue. They give assurance that the problems under investigation will be attacked with ability, courage and thoroughness.

The saying is familiar to every one that he who makes two blades of grass grow where one grew before, is a public benefactor. How much greater the benefaction if grass be made to grow on lands which formerly produced nothing. The race is dependent on the soil and any means by which food can be wrung from the lands that now lie waste is of vast importance.

In connection with the current discussion regarding the utilization and conservation of natural resources, one of the most important departments with the largest possibilities is the development of new varieties of animal and plant growth which will increase the available food product of the world. It is probable that with the aid of scientific methods a great deal can be accomplished in this direction. While irrigation has done and can do wonders to increase the product of the arid regions, the area to which it is applicable is, after all, only an exceedingly small proportion of the whole. The extension of dry farming, so-called, has perhaps even greater possibilities than irrigation. It is said that Luther Burbank, the famous California horticulturist, has developed a species of spineless cactus. It is well known that a great proportion of the so-called desert lands of the southwest contain moisture enough to support a heavy growth of plants of the cactus family variety, and it is claimed that these can be developed into a product which will make an excellent food for stock. The Department of Agriculture, by the development of special varieties of wheat suitable for growth in arid regions lying farther north has in effect added many thousands, or hundreds of thousands, of acres to our available producing area.

There are also possibilities in the development and the distribution of animal species. A striking example is the introduction of reindeer in Alaska and Labrador. As is well known, the typical plant growth of the sub-Arctic region is moss. This growth, while unavailable as food for ordinary domestic animals, forms the staple food of the reindeer. It is stated that these animals thrive so well on this food, and increase so rapidly, that in Siberia the ordinary price of an adult reindeer is 25 cents. The greatest defect as regards habitability of the far northern region is, of course, the lack of food supply. The stocking with reindeer, therefore, of these far northern districts may add a very large area to the region habitable by man.

The recent disastrous floods in the valley of the Sacramento River in California, which have caused damages estimated at $2,000,000 to $5,000,000, joined with other facts pertaining to that stream, suggest some remarks on cooperative work for the control and utilization of the rivers of the country. The conditions affecting this particular stream have been brought to our attention by some statements recently made by Mr. F. H. Newell, M. Am. Soc. C. E., Director of the U. S. Reclamation Service. Mr. Newell called attention to the fact that for years past this river has been the subject of much study by various experts, but the studies have been devoted to special phases of stream control or utilization and often to narrowly local problems. Hereafter, Mr. Newell suggests, investigations along various lines should be consolidated under the direction of

> an efficient commission representing all interests, but restricted to none, in order that a comprehensive plan of development may be laid out and presented for the consideration of the State Legislature and of Congress.

The interest of the federal government in the Sacramento is "primarily on the side of navigation." The Sacramento and the San Joaquin and their tributaries form one of the many great American systems of inland waterways. The national government is also interested in the river through the national forests which cover large portions of its drainage area, particularly since, according to Mr. Newell, "the national forests have been set aside largely for their beneficial effects upon the rivers." Both the U. S. Reclamation Service and the U. S. Geological Survey are concerned, from various points of view, with the irrigation, stream flow and mining debris problems pertaining to the river. The Survey has been mapping reservoir sites, and also lands which may be irrigated and swamp and overflowed lands which may be reclaimed.

It appears from the statements made by Mr. Newell that reclamation districts are building levees along the river quite independently of each other and without any regard to the larger problems involved.

As to comprehensive studies of all the problems needing attention, Mr. Newell says in conclusion:

> The theory underlying the Reclamation Act by which the Government makes the surveys and examinations, advances the money and collects the cost from the landowners in ten annual installments, seems to provide the most businesslike way of carrying out any broad scheme of utilizing the arid or overflowed lands of the valley. To attain the best results broad plans must be made, beginning with the control of the national forests in the interest of water conservation. Next to this should come the construction of reservoirs to hold water for irrigation and to exercise a beneficial effect in modifying floods. Next, the general scheme of levees should be consistent with the best maintenance of the waterways. The drainage plans should be laid out with reference to the general system, the debris problem worked out on the basis of the latest information, and finally, the navigation channels maintained with the aid of the general works.

Probably no better illustration could be cited of the need of a radical change of plan of river study and improvement if there is ever to be realized the ideal of water utilization set forth in

the preliminary report of the Inland Waterways Commission:

> Every stream should be used to the utmost; every river system, from its headwaters in the forest to its mouth on the coast, is a single unit and should be treated as such.

Such unit treatment of the Sacramento, we presume, would involve water-power and municipal water-supplies, neither of which appear to have been mentioned by Mr. Newell.

Of course, the Sacramento is only one of many streams which need comprehensive study and improvement. So many are these streams and so great are some of the problems which they present that we can see no hope for unit treatment except by the cooperation of the nation, the several states and oftentimes cities and towns or irrigation, drainage and levee districts.

There will be many instances where the states, rather than the general government, will have to take the lead. This has been done by the State of New York in the river improvement, water-storage and water-power studies reviewed at length in our issue of Feb. 18. Those studies, commendable as they were, need to be broadened in both national and local directions, in order to secure ideal unit treatment from the headwaters to the mouth of the various streams of New York. Yet, as we have just intimated, and as we said more pointedly in our issue of Feb. 18:

> Pending the time when public sentiment has crystallized sufficiently to determine the relative parts which the nation and the several states will take in these matters it is the duty of each state to work out a program for the conservation and utilization of its own water resources.

If the states find it necessary to take the lead in such matters they will still doubtless be able to secure co-operative assistance from the national government. Such assistance has been given quite extensively for years past by the U. S. Geological Survey. A bill now before Congress, in the interests of navigation and forestry, provides for what is in effect co-operation of the U. S. Department of Agriculture, the U. S. Geological Survey and the one or more states in forestry conservation. Co-operative endeavor in all fields of human activity is the order of the day and so is the conservation of natural resources. We may therefore expect to see the nation, the several states and often more restricted areas as well, co-operating in the conservation and utilization of the great water resources of the country. To further this end nothing will be so powerful as a few well-conceived and executed examples of unit treatments of a single stream. Whether or not the Sacramento shall be one of the first streams for such treatment is not for us to say, but certainly few if any streams present a greater need of comprehensive control or greater possibilities of benefit therefrom.

The Reasons Why the Lock Plan for the Panama Canal is Preferable to the Sea Level Plan.

Three years ago, after long and careful investigation, the United States Government decided upon the lock plan of construction for the Panama Canal. This decision was made in conformity with the weight of the best engineering opinion.

We say this advisedly and with all due respect to the engineers on the International Board who, at that time, from such knowledge as was then available, favored the sea level plan. Secretary Taft and President Roosevelt, in making the final momentous decision in favor of the lock canal plan, accepted the opinion of such leaders of the engineering profession as Alfred Noble and Gen. Henry L. Abbot, who had given more thorough study to the Isthmian Canal problem than any engineers in this country or elsewhere; of Mr. F. P. Stearns, whose eminence in the field of hydraulic engineering is well known to every member of the profession; of Mr. Joseph M. Ripley, with his long experience at the Soo Canal and its great lock, by far the greatest canal in point of traffic and by far the greatest lock in size, to be found in the world; of Mr. Isham Randolph, the engineering creator of the Chicago Drainage Canal, which ranks with the greatest canals on the globe.

Besides these engineers, the lock canal plan was supported at that time by the engineers of the Isthmian Canal Commission, Gen. Peter C. Hains, Col. O. H. Ernst and B. M. Harrod, Past-President of the American Society of Civil Engineers. It also received the emphatic endorsement of Mr. John F. Stevens, then Chief Engineer of the canal work, and familiar, from close personal contact, with the problems involved in the question as to the relative merits of the two types.

We repeat, therefore, that Secretary Taft and President Roosevelt, in making the final momentous decision in favor of the lock canal plan three years ago, acted according to the best advice of the American engineering profession.

From that time to the present, we believe, the canal enterprise and the plan on which it is being carried out has deserved and has, we believe, received the general support and approval of American engineers.

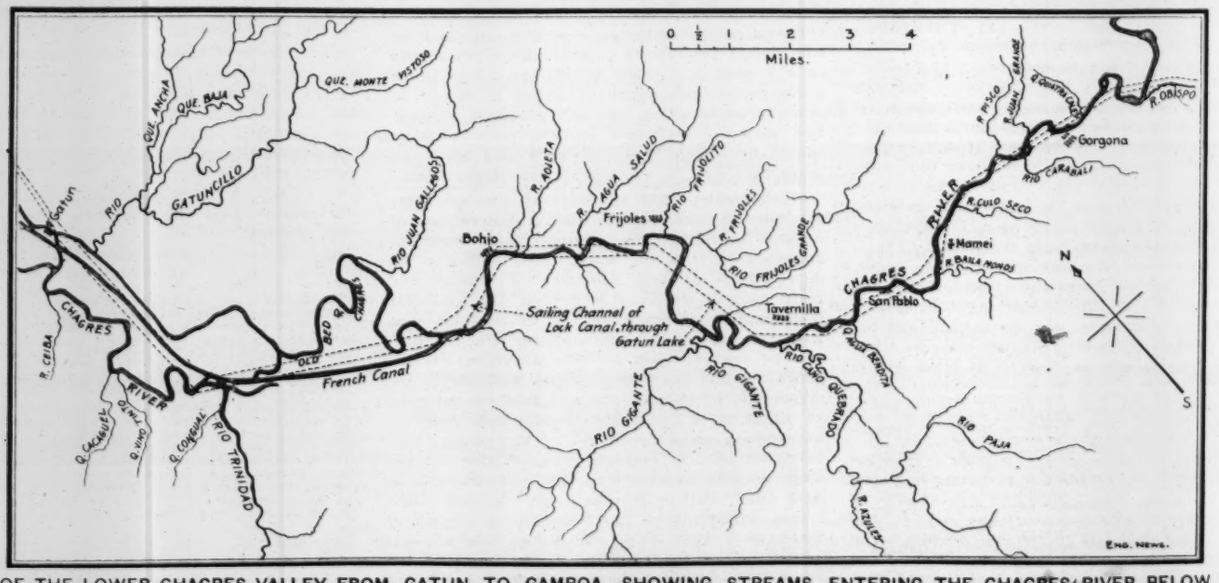
It is now nearly three years since the decision in favor of the lock plan was made, and those three years have witnessed progress in construction exceeding the most sanguine expectations then entertained. The engineers on the Isthmus, in personal contact with the work, familiar with all the problems involved and better fitted to judge than men can possibly be who have never seen the Isthmus, are [illegible] and all agreed that the lock plan of construction now being carried out is superior by far to the sea level project which was definitely abandoned three years ago.

No question of the correctness of this decision in favor of the lock canal plan was raised until some three months ago when a newspaper reporter happened to visit the Gatun dam site and noticed a place where some of the waste rock from the lock pits, dumped along one edge of the dam location, had settled considerably where it crossed the old river channel. We can learn from actual personal examination that this incident had absolutely no engineering significance. It made, however, an excellent sensational story. Telegraphed all over the United States and Europe, it was magnified into a catastrophe portending danger to the completed dam. Other stories followed on the heels of the first. The public was regaled with fanciful accounts of an underground lake beneath the Gatun dam site. The very precautions in testing the underground strata which the engineers were taking were construed as evidence that doubt existed as to the dam's safety.

From arousing public anxiety as to the safety of the Gatun dam, the newspapers proceeded to assail the lock plan of construction, and a hot campaign has been waged to bring about a change to the sea level plan. It is well to point out, however, that no American engineer of high professional standing has lent his name to this campaign. American engineers of reputation, it is almost needless to say, would not lend their names or their influence to further such an agitation, whatever their personal opinions might be.

However trivial the causes on which this agitation was based, and however lacking in expert authority, it is undeniable that public confidence in the canal work was shaken. It was therefore deemed advisable that Secretary Taft on his recent visit to the Isthmus should be accompanied by a board of engineers, selected from among the men of highest standing in the profession in all parts of the country. Also, since the stability of the Gatun dam was one of the main points in question, particular care was taken to select men of large experience in dam construction. While all the seven engineers of the Board rank high as experts in this field, Messrs. Arthur P. Davis and Jas. D. Schuyler particularly, rank second to no engineers anywhere as authorities on the construction of dams under varied conditions.

This Board of Engineers has just made its report of the results of its thorough examinations on the Isthmus. That report is published in full in this issue and constitutes a most emphatic



MAP OF THE LOWER CHAGRES VALLEY FROM GATUN TO GAMBOA, SHOWING STREAMS ENTERING THE CHAGRES RIVER BELOW THE GAMBOA DAM SITE.

endorsement of the lock canal plan now being carried o...

We believe that almost all engineers, even those who may have formed strong prejudices in favor of a sea level canal, will realize that the opinions of engineers of such high standing, formed by a study of actual conditions on the ground, must be accepted as authoritative and conclusive, particularly when taken in connection with the practically unanimous support of other competent engineers who have studied the work on the Isthmus.

We do fully appreciate, however, that many men, and even many engineers are honestly impressed with the current published arguments in favor of a sea level plan and against the canal with locks. We fully recognize that the sea-level idea appeals to the popular mind. The layman who knows nothing of engineering can understand the sea-level plan—or rather thinks he can—whereas not one man in a thousand has ever seen a canal lock, or has any idea of its operation. Indeed one might well suppose from current discussion that a canal lock was a new and fearsome invention which had never been tried. Actually, of course, it is one of the most ancient devices in engineering, its origin being ascribed to Leonardo da Vinci.

For the benefit of the man who wants to know the reasons why the lock canal plan is preferable to the sea-level plan at Panama, therefore, we have endeavored below to present some of these reasons in a simple non-technical way.

First, however, we must ask the reader to reject from his mind all that he has read concerning the canal work in newspapers and popular journals. So large a proportion of the statements thus circulated have been absolutely erroneous that any conclusions or opinions based on them are necessarily unreliable. And we include in this the public statements ascribed to various men in high position. To give a single illustration, we may take the statement widely published on Feb. 9, that a sea-level canal at Panama could be completed for less than the cost of the lock canal. The actual fact is that a sea-level canal would necessarily cost many millions of dollars more than the lock canal and would take a number of years longer to construct.

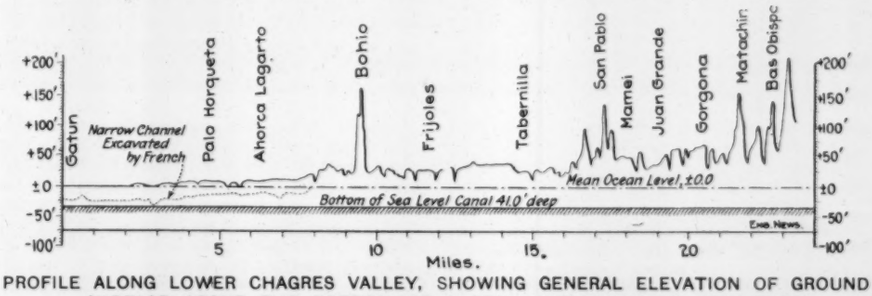
The engineers on the Isthmus make an estimate of over $100,000,000 as the minimum excess of cost of a sea-level canal over the lock canal *for construction alone.* This estimate does not include the cost of carrying on the work of Government and sanitation in the Canal Zone during the additional years which would be required to build a sea-level canal, an item amounting to perhaps $3,000,000 per annum. Neither does it include the interest during these additional years of delay on the amount which the nation has already invested in the canal. Furthermore, engineers on the Isthmus frankly say that the estimate above given for excess cost may be largely exceeded. There are problems in connection with a sea-level canal which have never been solved, and no engineer can say how many millions might be required to conquer some of the difficulties that face a sea-level cut across the Isthmus.

What are some of these difficulties? We will try to make them clear to the non-technical reader, who thinks of the sea-level canal as merely the task of digging a deep ditch all the way across the Isthmus. That looks stupendous but simple to the layman; and for a part of the way across the 40-mile neck of land which divides the oceans, this correctly describes the work. In the nine-mile section through the great Culebra cut, for example, with years enough of time and millions enough of money, the engineers can dig down through the rock and make the enormously greater excavation which the sea-level canal would require.

But it is in the long 25-mile stretch, from Gatun to Bas Obispo, through the swamps and along the course of the lower Chagres River that unsolved difficulties face the projector of a sea-level canal. How much digging will be required to create a 45-ft. deep sea-level canal over this section? Who knows how much material will flow into the cut from the sides? It is along this section that the Panama R. R. crosses a quagmire known as the Black Swamp, where piles 100 ft. long fail to find bottom. Here, in 1907, five hundred feet of track of the Panama R. R. suddenly disappeared. It must be plain to any one, even if wholly unfamiliar with engineering matters, that such material is most unfavorable through which to construct a cut of great depth. Remember, too, that the ground level steadily rises from Gatun, all the way along the Chagres valley, to Bas Obispo. In the swamp near Gatun the bottom of a sea-level canal would be some 50 ft. below the general ground level. Near Bohio the bottom would be some 70 ft. below this level. At Gamboa, where the Chagres River enters, the bottom of the river bed would be some 90 ft. above the canal bottom.

Can the reader imagine what would happen to this deep ditch, dug far below the level of the surrounding country and for much of the way through soft quagmires? Can he form an idea of the wash all along the sides of such a channel subjected as it would be to an enormous rainfall? Has he any conception of the results that attend a rainfall of 20 inches and over in a single month? Can he understand the radical difference between a deep ditch cut through the rainless desert of Suez and a far deeper ditch cut through the soft alluvial deposits of the lower Chagres valley?

As it happens, we are not without actual experience as to how rapidly nature acts in filling up a ditch through the Chagres swamps. A score



PROFILE ALONG LOWER CHAGRES VALLEY, SHOWING GENERAL ELEVATION OF GROUND SURFACE ABOVE THE BOTTOM OF A SEA LEVEL CANAL 41 FEET DEEP.

of years ago, the French dug a stretch of channel nearly 16 miles long, through this section, extending from the Atlantic coast inward nearly to Bohio. It was not very deep or broad (the original French sea-level project contemplated a canal only 29.5 ft. deep with a width of only 72 ft.) and the full depth was never reached on this completed channel. We said in commenting on this work in our issue of June 2, 1888:

> In view of the uncontrolled flow of the Chagres and the liability of this lower portion of the canal to damage from caving and silting, if this "open section" is ever to form part of an interoceanic canal, it will probably have to be all dredged out again years hence.

Exactly this thing has happened. Parts of this old French ditch now form the channel of the Chagres, which abandoned some miles of its old crooked course and a large part of the channel elsewhere has been filled with silt and mud, exactly as was predicted. And if this shallow ditch became so filled with deposit in a score of years, what would be the inflow of material into the huge ditch required by the present sea-level project, with its bottom 50 to 90 ft. below the level of this vast area of treacherous swamp?

If the reader perceives the importance of these facts he will understand, we are sure, the reason why the engineers who have given long study to the canal problem are accustomed to say that water is the enemy of the sea-level project but a friend of the lock plan with its great storage lake.

When the sea level plan was formulated as an engineering possibility, it was proposed to control the floods of the Chagres River by an enormous masonry dam at Gamboa, over 200 ft. high above its foundations and estimated to cost at least $6,000,000. This dam would create a great lake above it to store the floods of the Chagres and make its flow into the canal uniform. Provision must also be made, however, for disposing of the flow of the numerous rivers which enter the Chagres below Gamboa. Just how these could be disposed of is a problem which the projectors of a sea-level plan never satisfactorily solved. Many of these streams bring down heavy floods in the rainy season. If they were taken into the canal, they would be continually bringing into it vast quantities of alluvial deposits. The soft swamp mud through which most of them flow offers no foundation on which to support controlling works such as dams or weirs to regulate their flow into the canal. Further up the valley the ground is better, but on the other hand, the rivers are higher above the bed of the canal, and their control becomes correspondingly difficult. There are no less than 15 streams entering the Chagres in this upper section between Bohio and Gamboa.

We have said above that no one knows how much a sea-level canal might cost. No survey to determine the method of control of these subsidiary streams has been made. No exploration has been made of possible sites for dams. The probabilities are that for nearly 25 miles, from the Atlantic Ocean all the way along the Chagres valley, it would be necessary to build not one canal but three canals, viz., the channel for navigation and an artificial diversion channel for the rivers on either side.

We might give many more particulars regarding the difficulties involved in dealing with these Isthmian rivers; but we pass to the most important point, viz., the bearing of these difficulties on the usefulness of the completed canal. Engineers can build a sea-level canal if the nation wants to wait enough additional years and spend enough additional millions. That we have never denied. But engineers cannot accomplish the impossible, and the sea-level canal if completed would have its usefulness impaired by these difficulties, which we have briefly explained above. Constant dredging would be necessary all along this 25-mile section from Gatun to Gamboa to keep the channel clear. Vessels passing through the canal would have to dodge the dredges and scows. Further, the currents and cross-currents set up in the channel by the entering streams would be likely to cause much difficulty in the steering of vessels.

We have said above that there is a certain popular support for the sea-level project. Question the man who says he believes in the sea-level plan, however, and you find that what he really wants is the best canal that can be built. He assumes that the lock-canal plan is adopted because it costs less and can be completed in shorter time. We trust the above explanation may give a hint, even to the non-technical reader, that the lock canal will be actually better and more convenient for ocean traffic than the sea-level canal projected. We cannot do better at this point than to quote one of the ablest engineers who has had to do with the canal enterprise, Gen. Henry L. Abbot, of the Corps of Engineers. In his well-known book on the canal, published two years ago Gen. Abbot said:

> With abundant financial resources and unlimited time for construction it may be considered "feasible" from an engineering point of view to construct a sea level canal, but when completed it must always remain inferior as a transit route to the lake type adopted.

Gen. Abbot's opinion is also the opinion of many other engineers who have made careful study of the problems on the Isthmus itself and who have been directly engaged in the work, the

former Chief Engineer, Mr. John H. Stevens, for example. Further, we could name not a few engineers who have formed strong prejudices in favor of a sea-level plan, but who reversed their opinions when they went to the Isthmus and made personal study of conditions there.

In the current comparisons of the sea-level and lock plans, it is frequently declared that all the work thus far done on the present plan would be useful in the sea-level plan. This is absolutely false. Millions of dollars have been expended upon the present adopted plan which would be wholly wasted if a change to the sea-level plan were made.

Again, the current discussion of the two projects in the newspaper press goes back all the time to the conditions existing when the International Board of Consulting Engineers reported three years ago. But the relative standing of the two projects is not at all what it was at that time. The sea-level project is still a mere plan on paper with all the difficulties and unknown elements which then existed, still unsolved. The lock canal plan on the other hand has made large progress toward completion. Many of its features which were more or less vague three years ago have now taken definite form. We may allude to two or three about which hottest discussion has raged.

Foremost of these, of course, is the Gatun dam. Three years ago, enough was known concerning the conditions at the site of the Gatun dam to satisfy the best experts on dam construction of its entire practicability and safety. But some bridge engineers and railway engineers and pseudo-engineers who knew very little about dams, thought it necessary to criticise its design. Since that time the material at the Gatun dam site has been more thoroughly investigated probably than was ever done prior to the building of any dam in the world. There have been test pits dug and borings made galore. Furthermore a good deal has been done on the actual construction of the dam.

We can testify from actual personal observation and study of the dam site and of the borings and pits that the Gatun dam will be as safe and permanent as any structure ever reared by man. Yet it is needless for us to say this in view of the emphatic approval of the dam by the Board of Engineers which has just visited the Isthmus, and which includes the highest authorities on dam construction that could be found anywhere. We may assure our readers further that the criticisms of the Gatun dam have emanated almost wholly from irresponsible newspapers. We have seen not a single criticism of the safety of the dam since its foundations have been explored, emanating from any hydraulic engineer of standing. Nor has there been any criticism from any source containing matter worth any serious attention from engineers.

A second (and comparatively minor matter) is the cost of the land to be submerged by Gatun lake. One argument of the sea-level canal advocates was that the United States would be mulcted in heavy damages in the condemnation of this land and they set the figure at not less than $20,000,000. In contrast with this, the estimate made in 1906 by the engineers responsible for the lock plan of construction was $300,000. It is interesting to record that title to all this land has now been secured by the United States and the total cost is under $285,000.

And now turning to the locks themselves, we wish to point out that the sea-level agitation is based on a fallacy. That fallacy is the idea that there is something objectionable or dangerous to a vessel in passing through a canal lock. Why should there be? The delay of a couple of hours or so required to pass through the locks is no argument against the lock system, for this delay—in itself a mere trifle in the course of a voyage lasting two weeks or more—will be more than made up by the faster time which ships can make when sailing through the broad open channels of Lake Gatun, as compared with the necessarily slow rate of sailing through a sea-level canal.

As for the danger, it is well known, of course, that every ship which passes through the Manchester ship canal has to pass through four locks and that in our own Soo canal, the greatest lock in the world passes a traffic more than double in volume that which passes through the Suez canal and made up of vessels far larger in average size. In all the years that these two great canals have been in operation there has never been an accident in a lock sufficiently serious to block the canal. On the other hand, there have been several accidents in the narrow channels of the Great Lakes where wrecked vessels have entirely blocked all traffic. This is another reason why the lock canal with its broad channels and free sailing will be safer for vessels and less liable to obstruction than the deep narrow ditch over 40 miles long which constitutes the ideal of the sea-level advocates.

These arguments, we well know, have been before presented; but they will bear repetition in view of the remarkable attacks on the lock canal plan which have been recently spread before the public.

Besides this, however, it is important to note the great work that has been done in development of the lock designs. We have spent considerable time carefully studying the designs for these locks, their gates and their operating machinery, which have been worked out with painstaking care by the very competent staff of engineers on the Isthmus. In simplicity, in effectiveness and in provision for every possible safeguard to insure the absolute safety of the vessels using the canal, the locks to be built at Panama will far surpass any ever constructed.

We may not take space at this time to describe the locks in detail; but we may record that by the introduction of powerful electric towing machinery the movement of vessels in the locks will be subject to absolute control, and the chances of collision of the vessel with the lock walls or with the gates will be reduced almost to the vanishing point. This, however, is only the first safeguard. Elaborate provision of guard gates is made to further insure the main gates against any possibility of accident. The locks are in duplicate throughout, so that if one lock should be by any possibility disabled or require closing for repairs, the other lock will be available for traffic. Finally in case by any contingency impossible to foresee, the upper lock gates should be destroyed and a passage outward opened for the waters of Lake Gatun, an emergency dam is provided which can promptly be brought into position to arrest the outflow from the lake.

But the supporters of the sea-level plan fall back on the earthquake argument. They argue that Panama is an earthquake country, and that the first earthquake there will destroy the locks, whereas the sea-level canal is assured to be earthquake-proof. It is well to inquire what are the exact facts in this matter. First, those who imagine severe earthquakes to be frequent in Panama would do well to carefully study some of the unstable pieces of masonry in ruined churches and other structures in Panama City. Some of these pieces of unstable masonry have been standing since the 18th century. We have recently personally examined some of these structures, and they seem to offer conclusive proof that for over a century at least, Panama has suffered no severe earth shock.

On the other hand, it is of course true that at any time Panama, or any other region of the globe may be visited by a severe earthquake. Such a cataclysm it is conceivable might perhaps be severe enough to damage the walls of the concrete locks; but if it were violent enough for this, then it would probably also be violent enough to damage the great dam at Gamboa, which it must be remembered is a necessary part of any sea-level canal plan. And the rupture of the Gamboa dam would be pretty certain to wipe out all that part of a sea-level canal from Gamboa to the Atlantic.

But there are other timid persons who favor a sea-level canal because forsooth a lock canal is so vulnerable, you know, in time of war. It only needs an evil-disposed person with a stick of dynamite to destroy a canal lock!

Concerning this idea it may be remarked first that the amount of damage possible to a structure of the vast size of a Panama Canal lock by such high explosive as could be carried about one's person is greatly overrated in the popular mind. Of course an evil-disposed person with high explosive could do some damage and obstruct a lock canal in time of war, if locks were left unguarded and he were permitted to go where he pleased. But there will necessarily always be an operating force at the locks and in time of war or threats of war, it is safe to assume that great care will be used in admitting persons to the premises about the locks.

The fact is that an evil-disposed person at time of war could block a sea-level canal just as easily as a lock canal, and probably at less personal risk. Concealed in the swamps of the lower Chagres, such a person could plant a torpedo in the channel at his leisure, exploding it under the hull of the first large vessel which came along and the sunken ship would effectually block the passage.

Another argument brought forward is that the locks now built will forever limit the size of ships. These locks are 1,000 ft. in length, 110 ft. in width and over 40 ft. in depth. They will admit with large space to spare the huge merchant or naval vessel now building or projected. Most engineers best qualified to judge believe that the vessels likely to use the Panama Canal will not for generations to come equal the size of the present large Transatlantic liners. But if, and when, the demand ever does arise for the passage of larger vessels than the present huge locks will accommodate, it will be a perfectly simple and easy matter to build an additional flight of larger locks alongside the present ones, and provide for the necessities, whatever they may be, of that far-off generation. And such locks can be built then, for a small part of what it would cost now to make a sea-level canal.

And now in conclusion let us make a fair comparison of the two channels, when completed; on the one hand the lock canal with its great inland fresh water lake, and on the other hand the deep narrow ditch all the way across the Isthmus which would be required for a sea-level canal. We say a *narrow* ditch and the actual figures justify the word. The original sea-level canal, which De Lesseps started to build in the '80's, was designed with a depth of 29.5 ft. and a width of only 72 ft., too narrow by far to permit the passage of the largest vessels of the present day. The sea-level canal projected by the International Board of Consulting Engineers in 1906 was designed for a depth of 40 ft. and a width of 150 ft. in earth and 200 ft. in rock.

In contrast with this, the lock canal now being constructed at Panama will have channels 45 ft. in depth, throughout. The channel from deep water in the Atlantic to the Gatun locks will be nowhere less than 500 ft. in width. In the deep waters of Lake Gatun, there will be practically free sailing in channels 800 to 1,000 ft. or more in least width for a distance of 20 miles. Approaching the cut through the continental divide, the channel is 500 ft. in minimum width for four miles; and for nine miles through the Culebra cut, the channel has a clear bottom width of 300 ft.

Again, the sea-level canal as projected three years ago had not only a narrow but a winding channel. Of the total length of 50 miles, over 19 miles was made up of curves. In contrast with this the curvature on the lock canal is exceedingly small. The few curves are of very large radii and are located in broad channels, so that there is no liability of a ship going aground in passing them.

We may point out, too, the great value to a vessel of passage through the fresh water of Lake Gatun. The marine growths on her hull will be killed by the fresh water, and this will largely reduce her rate of fouling, increase her speed and render less frequent her cleaning in dry dock. The great area of deep water in the Lake will afford an ample anchorage ground in a fresh water basin, protected from winds by the surrounding hills, while the many islands

and projecting headlands in the Lake will free it from wave action.

We have set down these facts without prejudice or bias, and they are facts that the American people ought to know. From a narrow and selfish standpoint it might seem to be to the interest of engineers to advocate a sea-level canal at Panama. Such a canal would require more years and more money to construct than the canal now being built and would give more employment to engineers.

Such considerations, it is needless to say, have no weight with the class of American engineers from whom the Government has sought advice as to the plan of the canal. The engineers who have planned and are carrying out the canal works are conscientiously aiming to create that canal at Panama which will be the best for the world's shipping. And the public may have entire confidence that their task is being efficiently performed.

LETTERS TO THE EDITOR.

The First Iron Ship.

Sir: A paper by H. Penton was printed in Engineering News of Dec. 3, 1908, p. 623, under the title "The Oldest Iron Ship in the World." The "Iron Age" of Jan. 28, 1909, also mentions it, in the following words:

"The Oldest Iron Ship in the World" is a paper read before the Society of Naval Architects and Marine Engineers at the general meeting in New York, Nov. 19-20. The author, Henry Penton, refers to the United States man-of-war "Michigan," which is the oldest iron ship in the world, built 66 years ago by Stackhouse & Tomlinson, Pittsburg, Pa.

This would be 1843. I think he is mistaken. "Iron Ship Building," by John Grantham says:

The first iron vessel and the first ever put to sea was built by Harsboy Co., of Staffordshire, and called the "Arrow," after the name of the projector. The boat was completed at the end of 1821. She was put together in the dock.

The size of the boat is not given. This is 22 years before Mr. H. Penton names the "Michigan." In 1829 another steam vessel was built, 70 ft. long, 13 ft. beam, 6½ ft. deep—and so on during the years up to 1838. All of these were built and in use before the "Michigan."

This city, Wilmington, Del., built the first iron vessel constructed in the United States, named "William W. Welden" and built in 1841. It was a side-wheeler, for service between Wilmington and Philadelphia.

Joseph H. Springer.

629 West Fifth St., Wilmington, Del., Feb. 6, 1909.

Angles of Slopes for Earthwork Calculations.

Sir: In your issue of Sept. 13, 1906, are published tables for locating engineers compiled by Mr. C. P. Howard, M. Am. Soc. C. E. Among these are some tables for excavation and embankment calculated for level sections, and for transverse slopes per station of 100 ft. I have had frequent occasion to use these tables since their publication in your valuable journal in making estimates for grading, earthwork, etc.

In the field my usual method has been to take the angle of slope at right angles to the center line with a clinometer, recording the same in the field book in the form of an equation; that is $\frac{10}{75}$ + or — according to ground, etc. It has always been necessary before making an estimate from the profile to change the "transverse slope" in the tables to read in "angles," say 45°, 30°, etc., corresponding to the tables for quantity.

I have recently had some discussion among my assistants who have been using the tables as to the correct angles corresponding to the transverse slopes as given in Mr. Howard's tables. I write you for information as to the angles of slope of the ground corresponding to slopes of 10% to 50%.

Yours very truly,

Lincoln Nissley.

711 Madison St., Evanston, Ill.

[Extended tables of slope ratios and angles are given in Trautwine's "Pocket-Book." Those asked for by our correspondent are as follows:

10%	5° 43′	33⅓%	18° 26′
20%	11° 19′	40%	21° 48′
30%	16° 42′	50%	26° 34′

We referred the inquiry also to Mr. C. P. Howard, whose reply is given below.—Ed.]

Sir: The slope of ground in tables, given as 10 on 100; 20 on 100; etc., means that the ground rises (or falls) 10 ft. in 100, etc. This method of designating the slope was adopted because it has been our practice to take 5-ft. contour lines, plotting them on our maps, and the slope of ground can then be taken by a pair of dividers from the maps immediately. I have never made a practice of taking the slopes with a clinometer, but locate each contour on the ground and draw it in on the sheets in the field. We use an ordinary Locke level for this purpose (fastened on a 5-ft. stick), and a 10-ft. level-rod, as more particularly described in my discussion of the paper of Mr. F. Lavis in the Transactions, Am. Soc. C. E., vol. LIX., 1905, pages 149-155.

Yours truly,

C. P. Howard, M. Am. Soc. C. E.

L. S. & M. S. Ry., Cleveland, Ohio, Feb. 5, 1909.

Small-Scale Working Drawings.

Sir: I was much interested in your article in Engineering News of Jan. 28, dealing with photographic reduction of working drawings to a portable size. For the class of work which the writer of the article apparently had in mind, I should think the method would be of great value, but for the general run of work in a large machine shop the expense and especially the delay incurred by the photographic process would in many cases be serious drawbacks.

In my office I have obtained excellent results as to cost and convenience by the use of small-scale drawings on cards 8 × 10 ins. in size. I find that draftsmen readily become accustomed to drawing to a small scale, and no more time is required than for large-scale drawings. Some extra care is of course needed in locating dimension lines and figures and in making them legible, but this has not been found a serious difficulty. The cards are of bond paper which takes ink much better than tracing-cloth and gives excellent blue prints with an exposure about twice as long as that required for a tracing. The cards are quadrille ruled, with thin blue lines ⅛-in. apart, which make it possible to dispense almost entirely with the T-square and frequently with the scale as well. Cards are classified alphabetically according to the kind of work to which they apply, cards in each class being arranged numerically on regular office files of the "Shannon" type, so that any card may be selected and removed almost instantly; in fact, the facility with which the cards can be classified and indexed is one of their strongest recommendations.

The adaptability of small card drawings to a very wide range of work has been a revelation to me, and I believe a little experience with such drawings will astonish anyone not previously familiar with their advantages. Drawings on which minute accuracy is required, and original drawings of machines, on which the design must be worked out in detail, are of course made to better advantage on a large scale; but such drawings rarely need be duplicated for shop use, and often they need not even be traced.

Yours very respectfully,

Arthur H. Morse,

Chief Draftsman, The Portland Co.

Portland, Me., Feb. 9, 1909.

Notes and Queries.

Through a typographical error, the caption of Figs. 2 and 3 accompanying the article "A Novel Traction Engine" in our issue of Feb. 18, was made to read "A Caterpillar Engine Plowing and Hauling in California October Soil." This should read "Adobe" instead of "October."

A LARGE VANADIUM-STEEL ORDER has been placed by the Ford Motor Co., Detroit, Mich., with the United Steel Co., Canton, O. According to a statement of the latter, the order calls for 2,100 tons, comprising three different kinds of materials, respectively for springs, for parts that require case-hardening, and for forged parts, axles, gears, etc. The three steels show the following properties, when tested unannealed, as rolled:

	(1) Spring.	(2) C-H.	(3) Forge.
Ult. Tensile Str.	140,000	55,000	111,250
Elastic Lim.	94,800	36,200	82,500
Elongation	7¼%	29.2%	27%
Reduction of Area	31%	65.6%	64%

Steel (3) can be raised to practically double the strength quoted, by proper heat treatment, its elongation being reduced at the same time to 12%.

A SYSTEM OF STATE HIGHWAYS for California is advocated by the California Good Roads Association, of which Mr. Charles D. Daggett, of Pasadena, is President. A rough draft of a legislative bill to this end provides for the appointment of three commissioners by the Governor, with a salary of $3,000 each. It appears that the proposition is to have the roads constructed by the counties, after previous approval of the location by the commission, and then to have the roads maintained by the state. This plan, it appears, does not meet with the approval of Mr. James N. Gillett, Governor of California, who favors instead a bond issue of $18,000,000, which he thinks would construct about 3,000 miles of roads and give the state a "system" instead of many disconnected networks of roads, the degree of connection depending upon the action or failure to act of the various counties of the state.

The Report of the Coroner's Jury on the Chicago Tunnel Crib Disaster.

The burning of the superstructure of the temporary crib over a working shaft for a new intake tunnel at Chicago was described in our issue of Jan. 28. This disaster resulted in the loss of nearly 60 lives. The jury drawn by the coroner (consisting of six members) included three engineers (A. Bement, John Ewen and Benezette Williams), and in consequence of this the investigation was made in a more intelligent and competent manner than would be the case with a jury composed entirely of laymen. The conclusions also are of much greater importance and force.

The verdict was rendered on Feb. 17. It attributes the fire to the dynamite stored at the crib, which thus accounts for the rapid spread and intense heat, and also for the large death list, many of the victims having been suffocated by the noxious gases from the burning dynamite. No criminal responsibility is attached to the city, the engineers or the contractors; but it is shown that city ordinances were not observed and that dangerous conditions existed. Such conditions of neglect of ordinances and rules, and failure to enforce measures of safety exist too often on works of this kind. The methods of storing and handling the dynamite were especially open to criticism, although they were methods of common practice. The jury therefore presented recommendations as to precautions to be taken in carrying on the work in future, which recommendations may be applied also to many works of a similar character.

In view of the wide importance of the matter in its relation to construction works in general we give herewith the report of the jury practically in full.

The said George Cooksey and 46 unidentified dead came to their death from suffocation, shock and burns, received by reason of fire, resulting in the destruction of what is known as the temporary intermediate crib, located in Lake Michigan about 1½ miles off the shore from the foot of 68th St., said fire occurring about 8 a. m., Jan. 20, 1909.

1. The fire which caused this loss of life, consisted in the rapid combustion of dynamite present in a room used for the purpose of thawing said dynamite, and the burning of this dynamite generated smoke and suffocating fumes in great volume, and was also accompanied by the evolution of intense heat and flames, which set fire to the structure of the crib. Owing to the lack of evidence, we are unable to determine the cause of the initial fire which led to the burning of said dynamite, but it is our opinion that it resulted from conditions then existing in said thawing room, which were not only dangerous, but liable to cause such fire. From the evidence and testimony, we find the following conditions existed at said intermediate crib at the time of said fire:

2. The superstructure of said crib was of massive timber construction, two stories in height, carried by a cribbed foundation filled with stone, the whole built in pursuance of a contract between the city of Chicago and the Jackson & Corbett Co., entered into Feb. 19, 1907. This provided for the construction of Section 3, of what is known as the Southwest Land and Lake Tunnel,* which was to be built according to plans and specifications provided by the then Commissioner of Public Works, William L. O'Connell. The contract was later assigned to George W. Jackson, Incorporated. This contract expressly stipulates that:

In all the operations connected with the work herein specified, that are performed within the limits of the city of Chicago, all city ordinances controlling or limiting in any way the action of those engaged on the works or affecting the materials applied to them, must be strictly complied with.

3. Upon the lower floor of this crib were located two steam boilers, pumps, engine and electric generator, compressed air tank, elevator engine and various other apparatus employed in the execution of the work, among which was a steam heated room for the thawing of dynamite. From this lower floor of the crib a shaft extended to the tunnel below the lake. Communicating between the shore and the lower story of this crib structure, there was a cable tramway upon which buckets traversed for the conveyance of material and men. By means of an elevator running between the tunnel and the first and second floors of the crib structure, the excavated rock from the tunnel was raised to the second story, where it was either deposited in buckets and thereby transported to the shore, or else thrown into the lake.

*Engineering News, Nov. 14, 1907.

4. Upon the second floor of this crib structure were located sleeping quarters, kitchen and dining room for the accommodation of men employed in the execution of the work. All of these rooms were provided with exterior openings, except one.

5. There were several fire hose 50 ft. in length, connected to water lines by stop valves located at the junction of the hose with the lines, and these pipe lines were connected with two steam pumps. There were four of these connections upon the lower floor, three of them upon the second floor and one in the roof of the cupola of the crib structure. It also appears that there was an overflow, controlled by a valve in these water pipe lines, making it possible for the pumps to continue in action, circulating water through the pipe lines when the hose was not in use, thus preventing the freezing of same in cold weather. There were about 30 fire extinguishers, which were hung on walls at various points on both the upper and lower floors.

6. There were between 300 and 500 lbs. of dynamite in the thawing room at the time of the fire, where it had been placed for the purpose of thawing. This room measured approximately 5 × 6 ft. and 8 ft. high. Upon its floor were located five return steam coils, 4 ft. in length and covering more than half of the floor area. These coils were supplied at full boiler pressure from a pipe leading through the upper part of the room and connecting with the steam pipe from the boilers. The safety valves of said boilers were set to blow at 115 lbs. per sq. in., corresponding to a steam temperature of 347° F.

7. There was only one stairway communicating between the first and second stories of the crib structure, which was located a few feet east of the room used for the thawing of dynamite.

8. At the time of the fire there was an accumulation of rock in the lake, which had been removed from the tunnel excavation and there deposited. It projected some distance above the surface of the water and rested against the northeast side of the crib. Upon this the majority of the survivors found refuge.

9. The openings through the exterior walls of the structure were approximately 24 in number, about equally divided between the first and second stories.

10. There were telephone connections on the crib, communicating both with the shore office of George W. Jackson, Incorporated, and the system of the Chicago Telephone Co.

11. The men were employed in two shifts of 12 hours each, one-half being off duty while the other half were at work, the change of shifts occurring at approximately 8 a. m. and 8 p. m.

12. There was a rule prohibiting smoking in the crib structure, but it was not strictly enforced.

13. There were no boats or life rafts upon the crib, but there were 12 or 15 life preservers. It appears, however, that the tug "T. T. Morford" was usually in the vicinity, and that on the morning of the fire it was at the permanent crib located about 3,000 ft. northeast of the burned structure, and that it proceeded immediately thereto and rescued men from the rock pile as well as from the water and ice floes.

14. The men had not been drilled in the use of the fire apparatus, and there was no established system for its employment. As to whether the fire line was in serviceable condition at the time of the fire, it is impossible to determine, as the testimony on this point is contradictory. It appears, however, that the desirability of inaugurating a fire drill had been discussed by those in charge of the crib work.

15. An iron pipe 7 ins. diameter, resting on the bottom of the lake, leads from the compressors on shore to the crib structure, thence to the tunnel, for the purpose of supplying compressed air to the rock drills and other machinery, as an auxiliary to steam power.

Now, having before us the above mentioned facts, we find:

1. That the dynamite thawing room was very unsafe in both design and construction, though following a common method where large quantities are handled. That it was possible for the dynamite to ignite, resulting in burning or exploding, which could be caused by dynamite dropping through openings in the floor upon the hot steam pipes of the radiator, or by overheating of the room which might occur, as there was no thermometer employed to indicate temperature, or any provision made for its control. While we are unable to determine whether or not these causes led to the starting of the fire which consumed the crib, we, however, emphasize the fact that it was possible for it to have originated from either, and it is probable that one or the other was the incipient cause of ignition.

2. That the deceased came to their death by reason of suffocation and intense heat previous to the actual ignition of their clothing, and that it was due to the burning of dynamite in the thawing room, the combustion of which dynamite in addition to the extreme heat, rapidly generated noxious gases in very great volume.

3. That there was a sufficient number of fire hose connections, but the main supplying these being but 2 ins. in diameter, was not of a capacity to supply the total number of hose connections attached to it. Provision appears to have been made to prevent freezing of the water in these pipe lines, and under conditions of a fire involving only the wood work the system would have been adequate, but with a fire involving several hundred pounds of dynamite, neither the use of this water system nor one of greater capacity would have prevented the catastrophe. Evidence shows that strenuous efforts were made by the men on the first floor to apply water to the fire with the fire apparatus, but owing to the rapidity of the combustion of dynamite and the resulting suffocating smoke, fumes, and great heat therefrom, it was impossible for the men to remain long enough in the vicinity thereof to do so. A like statement applies to the use of the fire extinguishers.

4. That the aerial tramway communicating between the crib and the shore offered no means of escape in this instance, and would be of limited value in the case of any serious fire at the crib, because of its low carrying capacity, due to slow speed at which it necessarily runs, and also to the fact that the wheel around which the power cable traveled at the crib end, would necessarily be rendered inoperative in case of such fire.

5. That boats or life rafts could not be depended upon as a practicable safeguard in all emergencies, because in case of severe storms or the gorging of ice about the crib, they could not have been used. They would, however, be of service under favorable weather conditions and should have been provided.

6. That the various doors, windows and other openings in the crib structure in the interior, as well as the exterior, walls were ample and well arranged, with the exception of one of the sleeping rooms occupied by laborers.

7. That the rock pile adjoining the crib structure was formed by rock excavated from the tunnel, which had been dumped from the crib. It appears that at the time of the fire the rock pile had arisen from the surface of the lake and afforded a refuge for the men who were able to reach it.

8. That this tunnel, which is to be used for the water supply for the southwest section of the city, is 16 ft. in diameter, built through live rock, about 165 ft. below the water line; that it was being built east and west from the foot of the shaft and that it required about 250 lbs. of dynamite to make a shot in each heading. The contractors had established a place for the storing and keeping of dynamite in the lake, about 500 ft. from the shore at 73d St., and in the establishment and maintenance of such place of storage, complied with all the regulations, municipal, state and federal. The evidence shows that the contractors from time to time transferred from the storage plant to the crib, only so much dynamite as required for daily use and consumption. Upon being delivered to the crib, it was placed in the thawing room, for the purpose of thawing preparatory to use.

We, the jury, after considering all the facts, find that none of the city officials, none of the officials of George W. Jackson, Incorporated, nor George W. Jackson, personally, were guilty of any criminal carelessness for existing conditions at the temporary intermediate crib.

We make the following recommendations in connection with this matter:

1. That a commission of competent men be appointed by the Mayor to determine the best methods for the handling, care and treatment of dynamite and other explosives, and to formulate rules and regulations therefor; that upon the report of such commission, proper legislation be enacted covering the subject. In making these recommendations, we have in mind similar measures which are now in force in the city of New York for the regulation and handling of explosives in that city. Pending putting into force permanent regulations as recommended, we urge that the proper city officials take immediate action to insure safety in use and handling of explosives. This we consider absolutely necessary in view of the fact that methods equally as dangerous as those employed at the crib are now and have been in vogue for many years in connection with work carried on for and by the city of Chicago as well as by others.

In this connection we feel it incumbent upon us to mention favorably the studies and experiments made by Mr. George F. Samuels, of the Bureau of Engineering of Chicago, upon a thaw house, which was built upon the principle that no such house should be warmed by high pressure steam, nor have a radiator within the room itself, and that the steam used should be of a very low pressure, supplied to an indirect radiator placed exterior to the thawing room, as described in his testimony.* In addition to the foregoing recommendation, we believe that certain matters pertaining to the restoration of the crib and resumption of work thereat, should be considered. Therefore, we make the following suggestions:

2. That the housing of workmen in such reconstructed crib, where it becomes necessary to detain dynamite for the purpose of thawing, should be discontinued.

*The thaw house designed by Mr. Samuels was described and illustrated in our issue of June 25, 1908.—Ed. Eng. News.

3. That the electric wiring should b[illegible] conduit, according to the requirements for buildin[illegible] ected in the city of Chicago.

4. That the safety of men employed [illegible] should be first considered, and to that end the [illegible] mod rn devices designed for safety in building [illegible] truction be used.

5. That iron ladders be attached to th[illegible] erior of the crib at various places; also a life [illegible] e extended around the crib at a sufficient distanc[illegible] ve the surface of the lake so that men could g[illegible] and support themselves thereon. In making th[illegible] commendation, however, we have in mind the fa[illegible] in freezing weather these ladders and the life l[illegible] could be of little or no avail.

6. That a pipe line of adequate capa[illegible] for the requirements of the fire system be used, [illegible] that a fire department be organized among the m[illegible] ployed on the crib, but under the control of th[illegible] Marshal. And that the crib be equipped with hose, [illegible] extinguishers and such other apparatus as is d[illegible] necessary by the Fire Marshal.

7. That boats be kept at the crib, read[illegible] r service.

8. That the pipe line carrying compr[illegible] d air from the shore to the crib be made to enter [illegible] side of the crib below the floor thereof, so that in [illegible] of fire on the crib it would not be injured, but con[illegible] to supply air to the tunnel.

9. That two stairways between the fir[illegible] and second floor be provided.

10. That there be windows through the [illegible] terior walls of all rooms in the crib.

Car Fender and Wheelguard Tests.

The report to the Public Service Commission, First District, State of New York, by the Sub-Committee on Safety Devices and Accident Prevention Measures, recently issued, covers the tests made at Schenectady and Pittsburg in September, October and November, 1908, to determine the merits of the various fenders and wheelguards on the market and of those that any inventors might submit for trial. In these trials stretches of track owned by the General Electric Co. and the Westinghouse Electric and Manufacturing Co., respectively, were relaid with sections of cobblestone and asphalt pavement to approximate the conditions prevailing in New York city. Dummy figures were used. These were made, as to weight, shape and joints, as near like human figures as practicable. There were three general types—a 50-lb. boy, a 120-lb. woman and a 170-lb. man. These were clothed in typical garments, as it was found that the clothes had no little effect on the operation of the safety devices. Not only the life-saving qualities of each device were taken account of, but also other points of interest to the railways—the cost and ease of maintenance, the weight, the simplicity of design and the material of which it was built. It was considered necessary to make these tests as in the similar ones previously conducted by various interests, proper uniformity of conditions and requirements had not been maintained. In advance of the tests a set of rules was drawn up, and widely circulated, showing the classification of devices and standard conditions of the tests for all fenders or guards of a similar character.

In all, 38 projecting fenders and 29 wheelguards were tested. On these 67 devices 1,801 separate tests were made, with the dummies standing and prostrate in all positions in which a pedestrian would be apt to be hit.

As a result of these tests the sub-committee of the Commission made several recommendations concerning the use of such devices in New York City—the only territory under the jurisdiction of the Commission. Some of these are reprinted below:

Recommendations.

All cars in service under the jurisdiction of this Commission should be supplied with two automatic (mechanical drop) wheelguards, which must be attached to the trucks of all types of cars, this apparatus to be always maintained in a normal operative and life-saving condition, the tripping gate to be carried at a height of not more than 5 ins. and the apron of the wheelguard not more than 4 ins. above the rails.

All cars in service under the jurisdiction of this Commission in the Boroughs of The Bronx, Queens and Richmond, should also be equipped with a projecting type of fender always carried in a normal operating position with the outer edge of apron not more than 6

ins. above the r... and maintained in a thoroughly operative and life...ving condition.

All cars in ...vice in Brooklyn should also be equipped with pr...ting fenders, and should carry them in a normal oper...ng position with the outer edge of the apron not m... than 6 ins. above the rails, except that such fender ...ould be lifted up into a folded position by the mo...man without the latter leaving the platform when ... are operated in certain streets.

The reason fo... he recommendation that all cars be equipped with ...matically operated wheelguards, and only certain of ... cars under the jurisdiction of the Commission be ... supplied with a projecting type of fender (either ...matic or manually operated), is that it is not cons...ed that any one device will be sufficient when a ... is being operated at high speed. Cars operating at th... low speed necessary in congested districts are suffi...tly guarded by automatically operated wheelguards, ... cars operated through localities where congestion is ... sufficient to prevent high speed should be equipped w... both projecting fenders and wheelguards.

Even though an automatically-dropped fender is used, experience prov... that it must be carried at least 6 ins. above the pavement, and this height will not always permit it, when tripped, to drop to the pavement quickly enough to get under a body in a prostrate position. The manually-operated type of projecting fender must also be carried at the same height above the pavement as the automatic, so that if the motorman does not drop the fender to the pavement before reaching the victim, it also will pass over him allowing him to go under the wheels of the car, if they are not protected by a wheelguard; but both the automatic projecting fender and the manually-operated fender are efficient for catching a person struck in an upright position. Therefore, it is considered essential that the cars should be equipped with both wheelguards and fenders, wherever local conditions do not interfere with the use of fenders.

Finally, it should be fully understood that the effectiveness of the foregoing recommendations is wholly dependent upon the maintenance of the devices in a thoroughly operative and life-saving condition. If such requirements are not exacted of the operating railway companies, the results which the Commission desires to accomplish cannot be obtained.

The exhaustive trials, while not proving that any particular fender or wheelguard had reached a point of efficiency to warrant its preference over all others, has, without question, demonstrated that there are several available fenders and wheelguards far superior in actual operative merit to most of the contrivances used at present, and less expensive to maintain in a life-saving condition. Their adoption by street car companies, not only in New York City but all over the country, would, therefore, tend to lessen in a marked degree the appalling number of fatalities and serious accidents occurring on street railways.

It is only recently that the great increase of street car accidents has brought home to the management of railways, principally through the claim department, the necessity of using a fender and giving to it the same care and consideration as is given to other parts of the car equipment. When the companies shall have instilled into their employees' minds the idea that these forms of safety devices should not be carelessly handled, but should be maintained in as efficient operative condition as the controller, circuit breaker and other essential parts, they will undoubtedly have accomplished a great deal toward reducing the amount of damage claims which is yearly brought against them.

Experience with Various Types.

The tests have shown that car fenders and wheelguards are like any other class of machinery. They must be gradually developed, and then be tested many times in actual use before they are available for service. While some of the devices tested had merit, they lacked the development which would make them suitable for use upon the surface cars in Greater New York. Some devices were of such poor workmanship and improper material that practically no true result of their pickup qualities could be obtained. Others had an impossible number of working parts, such as sliding contacts and joints, which would require lubrication and protection from the weather, and thus increase the care necessary to keep them operative. Again, others were built with a large number of spiral springs which it would be impossible to maintain in a normal condition.

Most of such devices were the result of ideas gained from witnessing accidents, by people who did not have any conception of the requirements of a fender or wheelguard for practical railway use. It was therefore not surprising that the devices which withstood the hard usage given by the tests and made the best percentage of pick-ups, were those of manufacturers who have long been in the business and had developed their apparatus along the lines required by railway companies.

In this country long experience has shown that climate, speed, street pavement and other influencing factors prohibit, on account of excessive cost, the use of any wheelguard or fender which is to be carried close to the track in a fixed position, attached to either truck or car body.

Wheelguards and fenders with aprons made up entirely of fingers pressed to the pavement when in action did not give the results anticipated by inventors of devices of this class, as the fingers, when dropped upon cobble or other uneven pavement, invariably struck the dummy on the rebound and either pushed it along or mangled it among the fingers, tending to reduce greatly its ability to get the victim onto the apron.

The material used in the construction of the device bears directly on its ability to withstand the rough usage caused by continuous service and largely influences its capability as a life-saver. Malleable iron should be used for all fittings, hangers and every part possible, as it will withstand a severe blow without bending, to which fittings of steel and iron prove themselves subject. The latter sometimes got out of shape, were sprung or became inoperative without the fact being noticed until they were again called upon to act. If fittings are malleable, the design can be of less weight for the same strength. The aprons of wheelguards that gave the best results were those formed of steel with wood slats, and those comprising thin strap steel and malleable iron braces. The slats and straps are so placed that when broken they can be readily and cheaply replaced without removing the apron from the car.

It developed during the tests with the 120-lb. woman figure that the wearing apparel was often the cause of failure of the devices either to pick up the dummy or to remove it from the track. When the fender hit the figure and pushed it along the track, the clothing sometimes trailed on the pavement and got under the front of the device with the result that in a number of cases the body was drawn under the fender. This brought out the fact that any successful wheelguard needs to be pressed to the pavement by a pressure of not less than 100 lbs. at the time it is intended to pick up an object. Devices that were operated by springs, instead of depending upon gravity, demonstrated this by making successful pick-ups where the others failed.

It is desirable also to keep the apron of both fender and wheel-guard as narrow as possible because of the shaking which takes place when a car goes over special track work or faulty joints, as the device is suspended only at the back. Its tendency to shake is, of course, proportionate to its width, and is undesirable for the reason that it loosens the parts, thereby causing a rattle and noise, and deranges the mechanism, shortening the life of the entire apparatus.

As it is found essential that a device should be forced to the track and held there by a compression spring, those depending only upon their weight being practically useless, it is desirable that the apron of either fender or wheelguard, when in normal operating position, should be pressed upward against a spring in order to retain it as far as possible in a rigid position and to obviate the shaking effect referred to above.

The tests developed the facts that, with a projecting fender, the transverse width of an apron is a very important point, and, in order to catch and retain a person struck by it, the width must be proportionate to the car's speed.

Very few of the projecting fenders tested had adequate bumper or dash protection to prevent the victim receiving serious injury by coming in contact with these parts. Only in the case of a very limited number of those that had dash and bumper protection was it apparent that an effort had been made to prevent the well-known throwing-out effect.

Another point in connection with the apron of most projecting fenders, as well as wheelguards, is that a straight apron must be wider than one of scoop shape in order to retain a body. This is an important point to the railway companies, as by knowing the speed of their cars through districts where a projecting fender is to be used, as well as through a district where a pick-up type of wheelguard is to be employed, they can, by testing, ascertain the minimum width of apron necessary for either fender or wheelguard to hold a body.

In the selection of a fender or wheelguard, the railway manager must give consideration to the practicability of the device he adopts to suit the operating conditions governing his road, such as pavement, track, grades, short curves and curves that pass close to the sidewalk at corners where it is likely that a projecting fender would catch pedestrians. Also snow and ice must enter largely into his consideration, especially in the selection of a projecting fender or pick-up type of wheelguard.

The height that the device was to be carried above the track in the tests was not fixed by the Committee, but was left to the opinion of the participant. Therefore, the railway manager must give consideration to the distance each device was carried above the track to secure the efficiency marks obtained. In a number of cases the devices undoubtedly were placed lower than permissible on many roads by reason of the condition of the permanent way. The height above the rail should be governed by the local conditions prevailing in each individual case, and the pick-up efficiency, as shown in these tests, must be considered proportionate to the height at which the device was carried above the track.

The tests could not prove altogether the ability of the devices to withstand the rough usage of continuous car service, nor in many instances the picking-up qualities of those which, after a few trials, were damaged so as to be inoperative, and had to be withdrawn.

The tests, successful as they were, unfortunately proved no one fender superior in all its parts to the others; but the results, as shown by the ratings, speak for themselves.

FENDERS.—In considering the devices tested as described upon the foregoing pages, there are the "non-automatic projecting fenders" which are serviceable only for catching a person struck in a standing position, as the fender must normally be carried at a height above the track, which permanent way conditions seldom permit of being lower than 6 ins. In the event of a person being in a prostrate position between the tracks, the fender will pass over it, and if the car is not equipped with a wheelguard, the victim will get under the wheels.

While the same may happen with a "platform-operated projecting fender," which also must normally be carried the same distance above the track as the non-automatic, the motorman can in this case drop the fender to pick up the object upon the track if he sees it in time; but with the non-automatic, even though he sees the object sufficiently far ahead, he is helpless to prevent it from going under the car and depending upon the wheelguard to save life, unless he is able to control the speed of his car and bring it to a full stop in time to prevent striking the victim.

There were several fenders submitted which were intended as a "combination fender and wheelguard," or "primary and secondary fender" arranged to operate so that if the front or primary fender passes over a person, or fails to pick up, the wheelguard or secondary fender will have been dropped ready to receive the person by the lifting action of the front fender. This principle is certainly worthy of the time, attention and effort being given to it, as we believe the future will demand this form of safety device for use upon cars for city service. None of the devices of this character was sufficiently developed to warrant its use.

The "automatic projecting fenders," of which several were tested, are designed to be differently operated, and to accomplish a wide range of results. The fender with the projecting bar extending in front of the apron, coming in contact with an object, trips the mechanical device, which drops the apron ready to pick up the object. This form of fender is available for service only where congested street conditions do not prevail.

During snowstorms, when fenders of this class for this reason would be particularly useful, trouble would undoubtedly be experienced on account of the automatic trip being operated by snow upon the track. Although, at the present time, this type of fender is in use upon several roads in Canada where they have more frequent snow storms than are experienced in New York City, and we are told that little trouble is experienced from these conditions.

Also among the "automatically operated fenders," there are those having a front bar, which, coming in contact with an object, is intended to drop the fender, apply the emergency air application to the brakes, sand the track, and shut off the power by the automatic circuit breaker over the motorman's head, thereby bringing the car to a full stop in as short a distance as possible, all of this being done by mechanism such as springs, valves and trips. As the cars for which these fender tests were conducted would not permit of a device which attempted to perform so many functions, further consideration is not given to this class of device.

The platform-operated fender, i. e., one dropped by the motorman by pressing a plunger with his foot or pulling a lever, is a type which has long been in service, and therefore has reached the highest point of development. We believe it will gradually be superseded as the automatic type of fender becomes more fully developed, but only upon roads whose cars do not operate in congested districts.

The platform-operated fender has undoubtedly demonstrated its efficiency in picking up objects, provided the device is maintained as close to the track as permissible, and the motorman drops it in time before reaching the victim; but of course, it is dependent upon the human element for its result.

In the tests made the platform-operated tender was not dropped to the track for standing positions, and proved efficient. For prostrate positions the fender was dropped by the motorman before it reached the body, as it was considered that the test was one of the picking-up qualities of the device, and not of the alertness or temperament of the motorman.

The "Shearing Projecting Fender." Several of this type were offered for test. They are intended to be attached to the car body in a rigid position so as to extend in front of the dash and be maintained not more than 2

ins. above the track in order to be effective for city service, but pavement conditions in Greater New York will not permit this. This type of projecting fender might be found suitable for some classes of service, but it cannot be given consideration for use upon cars within the jurisdiction of this Commission.

WHEELGUARDS.—The "Pick-up Type of Wheelguard." This wheelguard, like the platform-operated projecting fender, has long been in service, not only in many parts of the United States, but has been very largely used in Europe with considerable success. It has recently been the subject of much successful development by its manufacturers reducing its heretofore rather expensive maintenance, and especially lessening the chattering effect of the apron caused by its form of suspension, which is entirely at the back, therefore making it susceptible to the damaging effects of jolting caused by the car passing over defective track or special work.

There is little difference between the mechanism of these wheelguards. They are all operated by releasing some form of catch which normally holds the apron above the track, though some of them proved more sensitive and rapid of action than others. But all wheelguards of this type are designed to have the apron attached to the pilot board of the truck, or to the car body, according to the idea of the inventor as to which method of suspension gives the best result.

In the opinion of your Sub-Committee, all types of wheelguards either upon single or double truck cars, must be suspended from the truck to attain proper results.

The apron is usually carried normally 4 ins. above the track, and is dropped by means of a mechanism which is operated by a swinging gate. This gate is carried 5 ins. from the lower edge, above the rails, and hung as far in front as the available space under the car will permit (which should not be less than 36 ins.). Upon an object striking the gate it actuates a mechanism dropping the apron against the track by strong compression springs that are absolutely essential to the successful operation of this form of device.

"Automatic Gravity Drop Wheelguard." This device is similar to the wheelguard described above, except that it is dropped to the track from its normal operating position by its own weight, instead of being aided by the action of a compression spring to accelerate its dropping effect and hold it to the track under pressure. The tests demonstrated that, except in the case of cars moving slowly, at about 5 to 6 mi. per hr., this form of device without compression springs was too slow in getting to the track after a body had come in contact with the tripping gate, and the apron was not kept in sufficiently close contact to the pavement to prevent the victim being drawn underneath the apron.

"Platform Trip Wheelguard." This wheelguard has the same characteristics as those described above, with the exception that it depends for its operation upon the motorman, who must press a plunger to actuate the mechanism which drops the apron from its normal position. Your Sub-Committee is of the opinion that a manually-operated wheelguard is not efficient, and that all wheelguards must be automatically operated.

"Shearing Automatic Mechanical Drop." This form of wheelguard differs materially in its action from those previously described, inasmuch as it is intended by its triangular shape to shear the object from the path of the car. This device, by its advantageous form of suspension and its being held in normal operating position against strong compression springs, together with its triangular form adding to its strength without increasing the weight, reduces maintenance to a minimum. Whether the results to the victim are more severe by being sheared from the track than if picked up, appears to your Sub-Committee, from the great number of tests made, to be purely a matter of the position in which the victim is when struck. The shearing wheelguard efficiently prevented the dummy from getting underneath the wheels of the car, as it was exceedingly rapid in operation and was pressed to the track by heavy compression springs, which hold the guard strongly against the pavement throughout its entire width. This was demonstrated to be a very necessary feature.

"Shearing Automatic Gravity Drop." The characteristics of this wheelguard are similar to those of the shearing automatic mechanical drop, except that it depends upon its own weight to drop it to the track from its normal operating position. This serious defect, upon which comment has been made throughout the report, shows a lack of development in the essential features necessary to make it a successful device.

"Shearing Non-Automatic." This device is intended to be carried in a fixed position attached to the truck of the car, and must be carried at a height of not more than 1 in. above the track to be effective.

This form of rigidly supported wheelguard has proved, in the experience for some years of the railway companies in Manhattan, to be seriously difficult to maintain in an efficient life-saving condition, and any form of wheelguard that must be maintained in a fixed position close to the track must be condemned.

The Cast-Iron Sluice Gates for the Fens Gate Chamber, Charles River Basin, Boston Mass.

By WALTON H. SEARS.*

The Charles River Basin Commission,† in connection with its work of forming a fresh water basin by constructing a dam across the Charles River between the cities of Boston and Cambridge, Mass., has designed in its engineering department, and is at present erecting, five rectangular cast-iron sluice gates for the gate chamber at the outlet of the Fens Pond into the Charles River Basin. A description of these gates and their operating machinery follows.

The gates are of the rising stem type, have a free opening 6 ft. 4 ins. wide by 7 ft. 8¼ ins. high, and are to control the flow of storm water and the overflow from sewers in time of storm. The pressure may be from either direction, the maximum for which the gates and machinery are designed being 15 ft. over the sill or invert.

The general arrangement of one unit is shown by Fig. 1. The gate, frame and guides are of cast iron, but the bearing faces of the gate and frame are of bronze of the following composition: 70 parts copper, 2 parts tin, 26 parts zinc, 2 parts lead. This material is hammered into dovetail grooves and afterward machined. All sliding surfaces are faced with bronze. The gate has a tongue extending along each edge which slides in a corresponding groove in the guide, as shown in Fig. 2, which is a typical section of frame, gate and guide. The tongue-and-groove guide the gate and, when operating under a reverse head, take the pressure. As the pressure is reversible, it is necessary to hold the gate against the frame when in the closed position. This is accomplished by means of a fixed bronze-faced wedge on the bottom of the gate, which engages the bronze face of a yoke on the frame, as shown in Fig. 1, and by eight adjustable bronze wedges on the gate, which engage corresponding fixed bronze wedges on the guides.

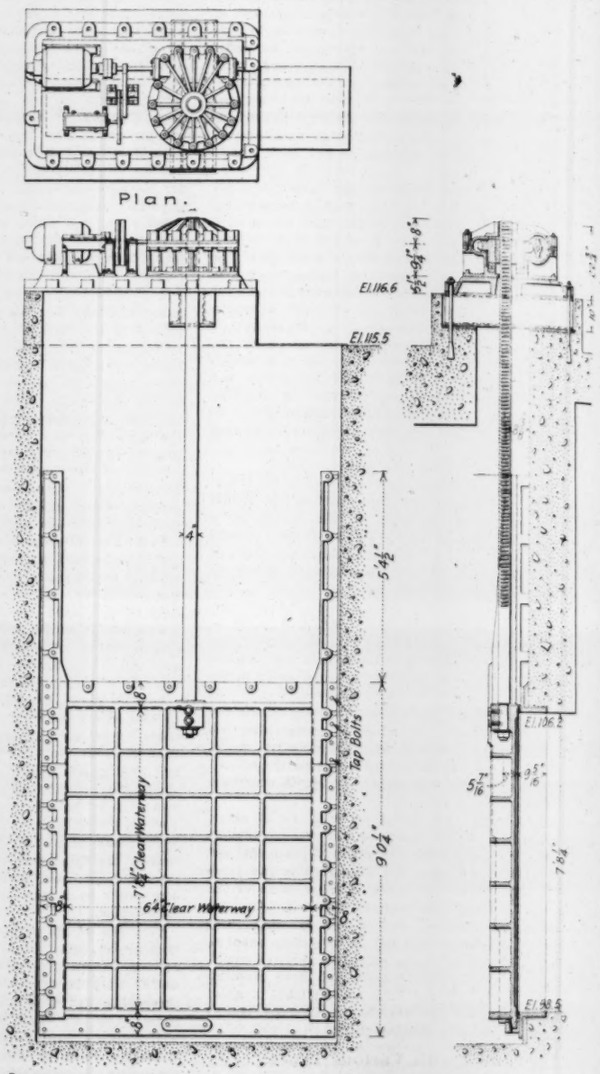


FIG. 1. GENERAL ARRANGEMENT OF SLUICE GATE FOR FENS GATE CHAMBER, CHARLES RIVER BASIN, BOSTON, MASS.

The operating stems are of rolled Tobin bronze, 4 ins. in diameter, turned down to 2⅝ ins. at the end where fitting into the gate and to 2 ins. at the extreme end where the lifting load is transmitted to the gate by a nut, held from backing off by a split pin. The stem is kept from turning in the fit in the gate by two special bolts screwing into the gate and extending through into the stem. The upper end of the stem has a square thread of ¾-in. pitch which engages with the operating nut.

This operating nut is of "Navy" bronze and is keyed to the inside of a Hindley worm wheel, the worm is direct-connected [illegible] armature shaft. Fig. 3 shows [illegible] the operating stand assembled. [illegible] raising the gate is taken by a [illegible] bearing and the thrust on the [illegible] taken to the operating stand thr[illegible] ings. The bearing area between [illegible] the top of the nut and the cap [illegible] stand is lubricated by a grease [illegible] The large ball bearing is kept [illegible] mersed in oil by an oil ring, as shown in Fig. 3, and the bearings of the worm shaft are lubricated by oil cups. The point where thorough lubrication is most important is of course the worm and this is provided for by keeping the casing partially filled with oil all the time so that the lower side of the worm dips continually in it. Mounted on the bedplate with the operating stand are the motor, the limit switch and the limit switch reduction gearing.

The motor is series wound for developing 7½ HP. on intermittent service and is guaranteed to allow stalling across the line for five minutes without injury. The speed of the motor is 640 r. p. m., which speed with the 20 to 1 reduction in the worm and wheel gives a gate speed of 2 ft. per min. The short duration of operation allows of the use of a 1-HP. motor frame. The motors were furnished by the General Electric Co. and are of their standard C R type, but are entirely enclosed, making a waterproof but not sub-

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†The scope of the work of this Commission and many of its details have been described in Engineering News of Jan. 12, 1905, p. 31; March 1, 1906, p. 243; March 15, 1906, p. 287; July 9, 1908, p. 27; Nov. 5, 1908, p. 498.

mergible machine. Moisture running along the shaft is intercepted before entering the frame by a leather washer.

The controlling apparatus was designed and furnished under contract by the Cutler-Hammer

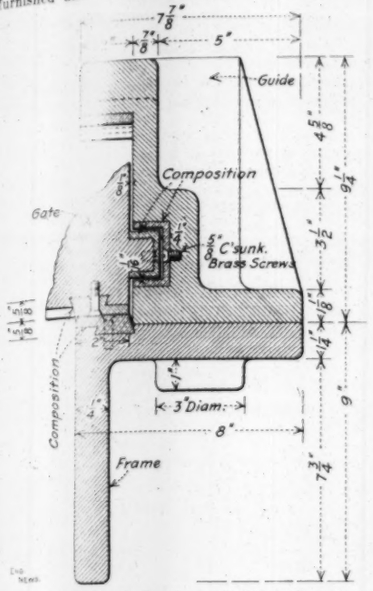

Fig. 2. Horizontal Section of Gate, Frame and Guide, Fens Gate Chamber, Charles River Basin.

Manufacturing Co. Each gate has an oil-immersed, reversing drum controller of very compact design, with automatic overload and limit-switch release. The first movement of the controller handle throws the motor directly

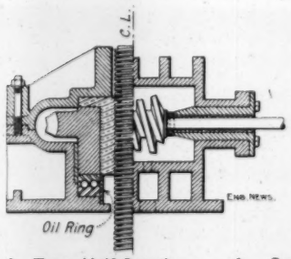

Fig. 3. Two Half-Sections of Operating Stand of Sluice Gate, Fens Gate Chamber, Charles River Basin.

across the leads but short circuits the overload release coil to take care of the starting current. The second and last movement removes this short circuit and the handle remains there until automatically released by overload or limit switch or until released by hand. In the complete installation there are five gates and the five controllers, together with a main switch, voltmeter and ammeter are mounted on one marbleized slate board.

The gates complete are being made by the Chapman Valve Manufacturing Co., of Indian Orchard, Mass., but are being erected by the Charles River Basin Commission. This Commission is composed of Hon. Henry D. Yerxa, Chairman, Hon. Joshua B. Holden and Gen. Edgar R. Champlin. William S. Youngman is Secretary and Hiram A. Miller, M. Am. Soc. C. E., is Chief Engineer.

ELECTRIC STEEL FURNACES, of the Stassano type have supplanted crucibles for the production of soft and tool steels at the plant of the Fraser Co., Bonn, Prussia, according to "L'Electricien." Two 1-ton, 250-HP. furnaces have been installed—one as a reserve unit. Three-phase current is furnished at 110 volts. The treatment of a single charge takes about 5 hours of which 1½ hours are required to remove sulphur and phosphorus and for reduction. The energy consumption is reported to be about 900 KW.-hrs. per ton of steel containing 0.08 to 0.18 carbon, 0.06% phosphorus and 0.03% sulphur. It is claimed that a low-grade material can be treated with good results.

Straight or Tapered Concrete Piles?

By CLARENCE S. HOWELL.*

It has been claimed that piles having tapering shape develop greater load capacity than those having lines parallel to their vertical centers. Timber piles, or the trees from which they are cut, grow under conditions of nature which give to them their familiar tapering shape, but it cannot be said that such shape is a factor in favor of their use. It is probable that this familiar shape may have fostered an impression that throughout the varied conditions and requirements of piling, of whatever style, no deviation should be made therefrom. On the other hand, attempts have been made to develop greater load capacity by driving timber piles with their greatest diameter downward. Without attempting a review of results from such practice, it will be sufficient to note that it sought to make use of the increased compression at the lowest stratum to be penetrated and to develop the load capacity and distribute the imposed loads, as far as possible, beneath the uppermost stratum, in rightful keeping with the fundamental principle underlying the use of pile foundations.

Certainly, in the manufacture of concrete piles, whether of the type cast before driving or that which employs a shell wherewith to mold such piles in place, it does not suggest good practice purposely to design the shape of piles so as to give them the extreme taper involved in the following comparison, and thus, contrary to the principle of pile foundations, to place, in great part, the weight upon the top stratum. It must be obvious that when tapered piles are driven upon close centers, by reason of their extreme taper, the compression due to displacement reaches its maximum very near the tops, and likewise that the natural compression of the earth at their points will not be greatly increased by their slight displacement. In the same sense, their small surface area offers small development of skin friction at or near their points, consequently making little use of the natural compression of the earth where it is generally greatest and most useful.

Of two types of tapered pile, that cast before driving, and that for which a shell is driven and then filled with concrete, the former is subject to the danger of fracture in driving, unless driven by a jetting process, and the latter is subject to reduction of its previously estimated load capacity, through disturbance of the shell before filling. The jetting process is well known to have no formula by which a load capacity can be estimated and is so dependent upon various other conditions that only an actual test load may be depended upon to establish the bearing value of each pile so driven. When a tapered shell is first driven, and afterward filled with concrete, especially when such a shell is tightly fitted upon a driving core, the upward thrust exerted in withdrawing the core will frequently pull the shell from the previously obtained penetration. The skin friction must be sufficient to counterbalance the upward thrust. Where the underlying strata are of a soft or semi-liquid nature it may be that sufficient friction is rarely obtained, and it is thus apparent that the penetration upon which the load capacity is estimated becomes subject to unknown alteration. Where piles of the cast type, having parallel sides driven, they would be subject to the same danger of fracture, though in probably less degree than the tapered pile of same type.

Probably the most successful method of placing the cast pile is that of first securing the required penetration by driving a shell and then placing therein a cast pile, the space surrounding the pile being filled with grout. It has not, up to this time, been found possible properly to drive the cast pile, except by this method. Apparently, therefore, the load capacity of concrete piles, when they are of tapered shape, is more or less obscure. This statement may also apply to straight cast piles, except when they are driven as described above.

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The advance of modern engineering work within the limits of the average city which so often may alter the ground water line and consequently impair the life of a foundation upon timber piles is just as liable to greatly disturb the upper stratum through which concrete piles have been driven. Removal of any considerable amount of the soil from the highly compressed and loaded area surrounding the tops of tapered piles must have an unquestionably detrimental effect upon the foundation resting thereon. Even were such piles driven to underlying hard stratum, by reason of the small cross sectional area at their points, they would not be safe as columns, unless reinforced with a considerable sectional area of steel. There does not appear to be any great difference in this respect between a foundation upon tapered piles and one upon a spread footing. Tapered piles would seem to have no other effect, considered from this standpoint, than the development of such a footing upon a limited area, or in other words, compression of the top stratum and corresponding decrease in the area required resulting from increased resistance.

Piles having a constant diameter necessarily cause equal displacement throughout their length and consequent compression, greater than tapered piles, at every point of their penetration. Since the natural compression generally increases in ratio to depth, by reason of its constant displacement and unvarying area of skin friction, the pile with sides parallel with its vertical center will develop its load capacity generally upon the lowest stratum into which it penetrates. Thus it makes use of probably the most important element entering into the distribution of loads from foundation piles, which element is almost entirely lost when piles having any considerable taper are used.

When tapered piles are driven through a comparatively hard stratum, underlaid by a softer stratum, the driving form, with its thin point will quickly penetrate to a considerable depth till compression around the upper section of the piles becomes so great as to require but few blows to develop the required load capacity. Under such conditions, might it not be proposed that to increase the taper would increase the ultimate value of the pile? Likewise, might it not be suggested that when driven under such conditions, to shorten the length of such piles would be advantageous, such suggestion resulting from the practically useless penetration of the slender form into soils of very soft texture upon which little compression is developed and insufficient skin friction to make any great use of the natural compression. It must be admitted that the tapered form of pile, especially when such taper is great, results in an improper development of load capacity and such shape does not suggest improvement over timber piles having less taper.

These facts are best seen from the comparison between a straight and tapered pile given in the accompanying table.

TABLE SHOWING PROPERTIES OF TAPERED AND STRAIGHT CONCRETE PILES.

Tapered pile: Diameter of head..... 20 ins.
Diameter of point..... 6 ins.
Length ..... 20 ft.
Straight pile: Diameter ..... 20 ins.
Length ..... 20 ft.

	Cross sectional area sq. ins.	Displacement. cu. ft.	Equivalent* compression %	Frictional area sq. in.	%
TAPERED PILE.					
Point	28.3	....	....	....	....
First 5 ft.	70.9	1.7	4.7	1,461	14.9
Second 5 ft.	132.7	3.5	9.8	2,120	21.6
Third 5 ft.	213.8	6.0	16.9	2,780	28.4
Fourth 5 ft.	314.2	9.0	25.8	3,440	35.1
Total.....	....	20.2	....	9,802	....
STRAIGHT PILE.					
For each 5 ft.....	314.2	10.9	30.9	3,770	25.0
Total.....	....	43.6	....	15,480	....

*Percentage given represents the amount of compression in a compressible area limited by a radius of 1.5 ft. from the center of pile, or 1,017.9 sq. ins. to a depth of 5 ft., equal to a volume of 61,000 cu. ins.

The scope of this article precludes the possibility of a discussion relative to the probable

load capacity of concrete piles driven under varying conditions and by various driving methods. The question of distribution of such loads, however, is a pertinent one when it appertains to pile foundations and the factor of safety involved in such instance should be materially greater when the imposed load is distributed upon upper hard stratum through the medium of tapered piles.

From the dimensions, displacements, etc., in this table it will be noted that while tapered piles of the dimensions given have more than 35% of their entire frictional area and nearly one-half of their displacement confined to the top 5 ft. of soil through which they are driven, distributing an equivalent percentage of their loads upon the top stratum, piles having sides parallel to their vertical centers would distribute the same percentage of their loads upon the uppermost stratum as upon the underlying stratum, except as the increased compression of the lower strata would increase the frictional bearing in both cases. As already mentioned, the latter pile would make a maximum use of the increased natural compression, while the tapered pile would not greatly add to its load capacity on account thereof.

Allowing the maximum compressive strength of concrete to be 2,000 lbs. per sq. in., and using a factor of safety of six, 333 lbs. would be a safe load per square inch when driven to hard underlying stratum. The area at the point of the tapered pile of dimensions given would therefore entitle it to an allowance, when driven to hard underlying stratum, equal to 9,423 lbs., requiring when driven under such conditions that if the imposed load be 25 tons, 40,577 lbs. thereof will be dependent upon friction for support. Since the displacement of the uppermost 5 ft. of length of this pile would be nearly one-half of its entire displacement and its frictional area for the same section more than 35% of its entire frictional area, it is apparent that the uppermost 5 ft. of soil penetrated, even when naturally not greatly compressed, would become sufficiently compressed for a maximum development of skin friction upon that stratum. A very large percentage of the frictional load would therefore be concentrated upon the first 5 ft. in depth below the head of such a pile.

It can only be noted with regret that where conditions would ordinarily require piles to be driven to a depth sufficiently great to develop a required load capacity in ratio to the penetration, the fundamental principle underlying all pile foundations is frequently disregarded and the development of resistance due to compression of the top stratum adopted as an advance in the use of concrete piles. Likewise, it must be noted, not without serious consideration, that in the estimation and limit of load capacities of piles, such a practice is given no greater restriction than the correct method of driving piles with sides parallel to their vertical centers.

The Purchase, Inspection and Use of Railway Fuel.

In discussions as to fuel consumption and fuel economy of locomotives, reference is made as a rule to the necessity of proper selection and inspection of the coal purchased. Some interesting points on the practical difficulties involved in this work on large railway systems were given at the January meeting of the Western Railway Club (Chicago) by Mr. Eugene McAuliffe, General Fuel Agent for the Rock Island Lines and the St. Louis & San Francisco Ry.

In the discussion following the paper it was suggested that when unsatisfactory coal is found on a locomotive the engineman should report it, so that the source of supply can be traced. It was pointed out, however, that such a method is practically useless. Mr. F. Slater, of the Chicago & Northwestern Ry., remarked that in his case coal is delivered by steamers and stored in a stock pile of 70,000 to 100,000 tons. If an engineman should complain of a load of coal supplied from this stock pile, it would not be possible to tell where that particular portion came from. Mr. P. H. Peck, of the Chicago & Western Indiana Ry., and Mr. M. K. Barnum, of the Chicgao, Burlington & Quincy Ry., both remarked that the same is true of coaling stations or coal chutes. Nobody can tell what particular car load of coal is contained in the various pockets from which the locomotives are supplied. Mr. Peck urged that the proper plan is to buy coal on specifications and to test samples from the cars before the coal is accepted or unloaded.

The following are some extracts from the paper by Mr. McAuliffe and from his remarks during the discussion:

The best inspection is that which can command on the part of the inspector the whole force of the producers' organization to the end that good coal will be produced. It is not the proper thing for the purchaser to employ men to stand around mines looking at the coal that goes into the car. That means that the buyer is furnishing superintendence that the seller should provide and pay for. There is a disposition on the part of the producer to lean on the railway fuel inspector. There should be but one head to a coal business and the railway fuel inspector should not be a necessity, at least as far as exacting contract grade of fuel is concerned.

In the ordinary railway fuel department, whether handled by the fuel agent, purchasing agent or general manager, one inspector is assigned to a territory covering possibly four or five states. The coal inspector then accomplishes practically nothing. I prefer to pick out the mine superintendent who is guilty of the greater number of defections and give him a reasonable chance to bring his work up to the proper standard; or get out from him entirely, and his business. In other words, give the business to the men who will furnish and put into the car the class of coal the contract calls for.

Mr. Henderson's book on "Locomotive Operation" has a table of coals ranging from 14,000 B.T.U. to 6,600 B.T.U. I submit that coal containing only 6,600 B.T.U. will not take a freight or passenger engine over a division.

Some years ago on the Northern Pacific Ry. we tried to work with Dakota lignite that runs about 8,500. The railway built locomotives with combustion chambers, put in brick arches, put on stacks wonderfully and fearfully designed and proportioned, and they discharged motive power men (and firemen incidentally), because they could not get service out of 8,500 B.T.U. coal. Today they are buying coal in Illinois and hauling it up to Dakota, although they have found better lignite than they had then.

On our system we are burning coals purchased in Illinois that rank materially below other coals, and with better results, for the simple reason that the former do not produce a clinker or a refractory ash. The fact is, that if you get a coal that runs as high as 12,100 B.T.U. and you can keep a reasonably fair fire and get air to it, you will not have very much difficulty in getting steam. But if you take coal that runs 15,500 B.T.U. and it is 4% to 6% ash that makes clinkers, you will have trouble before you get over the division.

The question of B.T.U. is not the only one. Some coal ranking high in this respect forms a clinker that clogs the grates, preventing proper ingress of air and perhaps causing an entire failure. I am buying in Alabama, for the St. Louis & San Francisco Ry., a coal that runs 18% ash, and we rarely have an engine failure for the reason that the ash does not make a refractory clinker.

I have recently been a party to the matter of attempting to reduce engine failures primarily charged to coal. Stopped up flues are factors in the engine failure line. A remarkable thing about the coal business is that we have the fewest complaints where we get the poorest coal. We are buying semi-lignite coal in Texas. It runs about 12,200 B.T.U., and we do not have as many engine failures as we have with some coals that run 14,300 B.T.U. We are buying coal unscreened in certain districts, for instance, carrying 12 to 15% ash, that is less productive of failures than screened coal that is higher in B. T. U. and does not carry 6% ash.

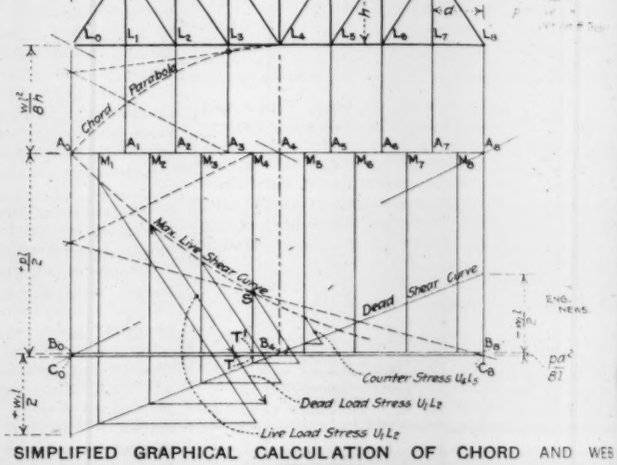
The only fuel performance sheet the railway can make today is to find out the total number of tons used during the month and divide that by the number of ton-miles hauled during that time and get the system-situation. We can get that reasonably close, but the individual locomotive, or locomotive-man situa[tion]. If some of the people who get out th[e] motive performance reports would [to] analyze one of them, they never wo[uld]

A change from one kind of coal to [an]other will cause trouble where the firemen are used to [one] if the latter is of superior quality. [Such] fluctuations we sometimes take thro[ugh] tives that are burning Alabama coal [in] Texas; take them out of a district [with] 36% volatile matter, and 50% fixed carb[on] in a district where the coal runs 70% [fixed] about 16% volatile matter. We never [change] in front ends or grates, but we do not [change] are used to the Alabama coal with [the] Texas, we employ men who are used [to] The engines are not nearly as sensitive [to] tions in the character of fuel furnished [by] the crews.

A Simplified Method for the Graphical Calculation of Bridge Trusses.

By J. H. GRIFFITH,* Assoc. M. Am. Soc. C. E.

The theory of stresses in bridge trusses under uniform loading has been so perfected that it is hardly possible to contribute much to the subject, but it is believed that the following proposed method and manner of arrangement of



SIMPLIFIED GRAPHICAL CALCULATION OF CHORD AND WEB STRESSES.
(J. H. Griffith, Assoc. M. Am. Soc. C. E.)

lines will give a considerable increase in efficiency to the ordinary graphical construction given in the text-books. These latter arrangements are essentially academic in presentation, and are not practicable for use by the routine bridge-office designer who, by a few slide-rule calculations, can compute all the stresses in a quarter of the time required to draw the usual series of diagrams.

The arrangement of lines proposed in the following is general in application to dead-load, live-load and engine-excess chord stresses in parallel-chord trusses of constant or variable panel length; to dead-load web stresses in trusses of constant or variable panel length; to live-load web stresses in trusses of variable panel lengths within limits of error which are negligible for such variation in panel length as is compatible with good practice.

The process and arrangement being an abridgment of the older method requires a partial restatement of the latter for continuity of discussion. It is based upon a simple law of the parabola which will be clear to the computer without further discussion.

CHORD STRESSES.—Referring to the figure herewith, in which for simplicity of treatment a bridge truss of uniform panel length has been taken: Lay off the full-load chord stress in middle panel, or full-load moment divided by depth, $\frac{wl^2}{8h}$, as shown, downward below the truss, draw-

*Assistant Professor of Civil Engineering, University of Michigan, Ann Arbor, Mich.

ing the line A_3 A_4. Suppose the stresses in members U_3 U_4 and L_3 L_4 are desired, the moment centers for which are at U_3 and L_3 respectively. To get value of stress directly without recourse to the usual subdivision of the vertical L_0 A_0 draw a line through A_3 parallel to L_0 A_4 (dividing the midspan point of A_0 A_3) and get the intercept upon L_0 A_0. From this point draw a line to L_3. The intercept upon L_3 A_3 gives the stress, the point of intersection being a point on the parabola representing the chord stresses.

The advantage is that the necessary ordinate to the stress parabola may be found with a minimum of lines, and that the method is true for varying panel lengths. Other stresses may be obtained similarly, as well as live-load and engine-excess chord stresses.

A parallel rule is the most advantageous instrument to use, no lines being required except intercepts on the verticals. This method would be well adapted to computations under such specifications as those of the Pennsylvania Lines West of Pittsburg, where the loading is 5,000 lbs. per ft. plus 50,000 lbs. engine-excess.

WEB STRESSES.—To get live-load shears erect the middle vertical for each panel and by the following procedure get the intercept of the shear parabola on the necessary vertical. A base line is drawn a distance ½ pl. below A_0 A_8, giving the line C_0 C_8. The value ½ pl. of course is the maximum shear. Parallel to the diagonal C_1 A_1 draw a line through some mid-panel point for which the shear is desired, as say, M_4, and get its intercept on the vertical A_0 C_0. Thence draw a line to C_8; its intersection with the vertical through M_4 gives a point S on the live shear or reaction parabola, which marks off the shear ST in the panel L_3 L_4. This, of course, is the shear on basis of a beam, which is amply accurate for truss calculation.

To get the shear as usually calculated, i. e., for full panel-loads applied at panel-points to the right of the section, we find in a similar manner as before the intersection of the shear parabola with the vertical M_8, and through this point draw a new base line B_0 B_8, which will give for example the shear ST'. The intercepts between this line and the shear parabola on the middle verticals are the shears as ordinarily calculated. The height C_8 B_8 is the difference between beam shear and conventional truss shear, and is equal to the shear produced by load on the right half of the end panel.

To combine these live-load shears with the dead-load shears, lay off the dead-load shear line through B_4 on B_0 B_8, draw lines parallel to the necessary truss diagonals, and as in the ordinary manner we have the subdivided live and dead-load web stresses, including the counter stresses.

The writer believes that while this method is merely a different statement of the older graphical method, it is a decided abridgment of the actual laying out of the constructions, and is almost mechanical in execution. Perhaps the only warrant for publication is that he continually notes the cumbersome methods in text-books for drawing parabolas and the equally cumbersome methods in vogue in the graphic calculations of the bridge office, and the inference is that the shorter and direct processes are unknown.

Notes From Engineering Schools.

STEVENS INSTITUTE OF TECHNOLOGY.—The annual alumni dinner was held at the Hotel Astor, New York City, on Feb. 19. There was an attendance of about 350. The toastmaster was Henry Torrance, Jr., M. Am. Soc. M. E., and the speakers were: President A. C. Humphreys; Mr. Alfred Noble, Past President Am. Soc. C. E., and a former member of the Panama Canal Commission; Col. H. G. Prout, Vice-President of the Union Switch & Signal Co.; Mr. John A. Bensel, Commissioner of the Board of Water Supply, New York City; and Col. Geo. Harvey. President Humphreys announced that at the next meeting of the Board of Trustees of Stevens Institute the following new trustees would be elected: Dr. H. S. Pritchett, President of the Carnegie Foundation for the Advancement of Teaching; John Aspinwall, President of the Fabrikoid Co., of Newburgh, N. Y.; Dr. D. S. Jacobus, Advisory Engineer, Babcock & Wilcox Co.; and Anson W. Burchard, Assistant to the President, General Electric Co.

AN EXPRESS TRAIN on the Delaware division of the Pennsylvania R. R. was wrecked Feb. 22 by a collision with two locomotives at Delmar, on the boundary line between Delaware and Maryland. Eight men were killed, none of whom were passengers. The train was the first section of the regular express leaving Philadelphia at 11.22 p. m. for Norfolk. The two locomotives were waiting for the two sections of the passenger train, but for some unexplained reason were on the main track. The railway officials are quoted as placing the blame for the accident with the fireman of one of the locomotives, who neglected to go back and flag the approaching express. The cars most seriously damaged were the baggage and the combination baggage and express. All the men in these cars were either killed or injured and the fire which followed entirely consuming the wreckage, delayed the recovery of their bodies.

THE STEAMER "PRESIDENTE ROCA," bound for the port of San Antonio, Argentine Republic, was burned and sank, according to reports of Feb. 20, off the east coast of that country. Reports state that 20 lives were lost, while the majority of the passengers and crew were rescued. The "Presidente Roca" was a mail boat 300 ft. long and with a beam of 41 ft.

AN EXPLOSION OF DYNAMITE on the Trans-Canadian railway construction work, 120 miles north of Nepigon, Ont., Feb. 20, resulted in the death of seven laborers. The charge exploded while being tamped.

AN EARTHQUAKE IN PERSIA in the Province of Luristan, Jan. 23, partially or wholly destroyed 60 villages and caused a loss of life estimated at from 5,000 to 6,000. Several villages, reports state, were completely engulfed and the comparatively small number of people killed is explained only by the fact that the region affected was sparsely populated. Practically all the cattle in this region were killed, the loss being placed at about 10,000 head. This earthquake was recorded by seismographs all over the world on Jan. 23, but has only recently been located on account of the isolation of the district where the disturbance centered.

CARS WITH SEPARATE EXIT DOORS were tried recently in the New York Subway, under an order of the Public Service Commission, as noted in our issue of July 16, 1908, p. 72. These cars are of the type recommended by the Commission's expert, B. J. Arnold, and were shown in Engineering News of March 5, 1908, p. 26. Their object is to facilitate the interchange of passengers at stations, in order to reduce the length of station stop and thus increase the number of trains per hour that can be operated over the line. Like the ordinary cars, they have a door in each end of each side, but these are to be used for entrance only and a separate exit door is provided about 4 ft. from each end-door. The attendant on the car platform operates the group of four doors in the adjoining ends of two cars, by means of levers. Cars of the ordinary type were converted to the new type and were made up into a regular 8-car train, so that actual observation would show how well they attained their object of reducing the length of station stop. Opinions differ as to the results. The operating company, the Interborough Rapid Transit Co., on whom the cost of altering the cars would fall, is reported as declaring the test a flat failure of the new car.

DAILY RECORDS OF STEAM SHOVEL EXCAVATION on the Panama Canal work were broken on Friday, Feb. 5. On that day 56 shovels in the Central Division excavated 73,442 cu. yds., an average of 1,311 cu. yds. per shovel. Of this amount, 17 shovels in the Empire District excavated an average of 1,530 cu. yds. each. The best previous daily record was made on Dec. 18, when 50 steam shovels in the Central Division excavated 70,087 cu. yds., an average of 1,402 cu. yds. per shovel. It is to be particularly noted that this work is done in a working day of only eight hours, out of which must be deducted time lost by moving shovels forward, blasting rocks too large for the shovel to handle, bringing in empty trains to replace full ones and making any minor repairs on the shovel, such as repairs of broken chins.

A STATE LICENSE FOR ENGINEERS AND SURveyors is called for by a bill pending in the Idaho Legislature. The bill requires all persons engaging in land surveying, topographic engineering, hydraulic and hydrographic engineering, construction and designing engineering or administrative irrigation engineering to pass an examination before a board of examining engineers and take out a license.

ELECTRIC DRIVE FOR PAPER MACHINES.—The first Fourdrinier paper making machine to be driven by individual electric motors on the various parts has just been placed in operation by the Gould Paper Co., at Lyons Falls, N. Y. This is claimed to be the first time that electric motors have been used to overcome the difficulties caused by the exacting speed requirements of a paper machine, and the results are said to show that this is superior to the complicated belt, shafting and gear systems previously used. Absolutely unvarying speed of motors must be maintained in order not to break the pulpy web in its passage from section to section of the machine.

It requires tedious adjustment to get the "draw" of paper between the several sections adjusted so that the web of paper will run the longest possible time without breaking. There is a considerable loss of time and material when a break occurs. It is difficult to maintain a continuous sheet of paper on account of the inconsistency of the stock, the irregularities of the mechanism, the elongation of the web due to pressing and the shrinkage in the drying process and in regulating the driving mechanism so as to allow for the contraction and expansion of the sheet of paper in passing over the machine.

Heretofore, uniform speed in the various steps of the process have been obtained by two accepted methods. Both involve complicated systems of shafts, countershafts and friction clutches, with cone pulleys for the fine adjustments of speed, and bevel gearing or long quarter-turn belts. These systems, in addition to their great complication, are open to several objections, as high construction and maintenance costs and danger to property and employees.

If constant speed were the only requirement, synchronous alternating-current motors might have been adopted to solve the problem. There is the added complication, however, that all classes of paper are not made at the same speed. At the Gould mill, for instance, newspaper is made at a high speed, and a heavy bag paper is also manufactured, which requires a very slow speed. Besides this the heavy starting torque required to start the couchers, presses, dryers and calenders makes the use of alternating-current motors impossible.

A system with multiple-speed, direct-current motors was worked out by the Standard Engineering Corporation, of Delaware, and the Crocker-Wheeler Co., of Ampere, N. J. The system can be operated at any speed to accommodate any kind of paper. The motors, once adjusted to uniform speed, are claimed to remain in adjustment indefinitely.

TRACK SHIFTING MACHINES on the Central Division of the Panama Canal work moved over half a million lin. ft., or 99.8 miles of track during the month of January. These machines are used on the dumps where the spoil from the Culebra cut is disposed of. The material is plowed off the Lidgerwood cars and is then leveled off by a spreader until the bank at one side of the track reaches a width of 10 ft. or so. The track shifting machine then begins at the end of the spur track running out on the dump and hoists the track in successive sections and throws it to one side to the edge of the dump.

MANGANESE STEEL was discussed in a paper read before the Western Society of Engineers on Feb. 17 by Mr. W. S. Potter, of Paterson, N. J. In regard to the proportion of manganese, it was shown that the characteristic strength and toughness due to this element are not obtained with less than 8%, and the content may run as high as 35%. As a rule, manganese steel contains from 11 to 13% of manganese, with 1 to 1.20% of carbon, 0.25 to 0.40% silicon, 0.06 to 0.11% phosphorus, and 0.02 to 0.06% sulphur. The steel has a high resistance to abrasive wear; in screens, it will last five or six times as long as ordinary steel if the abrasion is by sliding, or 80 times as long as the latter if the abrasion is that of the impact of ore or crushed stone falling from a height of several feet.

The use of manganese in steel castings and forgings was commenced some 30 years ago in England by Mr. Hadfield and his son, who found that with the higher percentages the product was very tough and strong and had a remarkable resistance to abrasion. The proportion of manganese mentioned in Hadfield's early patents as being suitable for track work castings was 11%. No heat treatment was mentioned in the early papers, but later such treatment was adopted. This consisted in heating the casting (at first slowly and then rapidly) to from 800° to 1,100° C., and immediately quenching it in water.

The rolling of manganese steel rails has so far been done only under Mr. Potter's process, the details of which have not been made. Considerable difficulty has been experienced in experiments upon this line, owing to the heat conditions, but Mr. Potter stated that under his process the steel is ductile while hot and can be rolled commercially. The steel is neither burned nor cracked. The product is tough and reliable, has a uni-

formly fine crystalline structure, and a much higher elastic limit than if rolled and finished in any other manner which he has tried.

Specifications for rails which will meet the drop-tests and standards of the American Railway Engineering and Maintenance of Way Association require tensile test bars cut from the cold rail to give a tensile strength above 100,000 lbs., elastic limit above 55,000 lbs., and elongation about 20%. Flat strips must bend (cold) 180° around a diameter twice the thickness of the specimen without evidence of failure. This test ensures the toughness which determines the wear value. The chemical composition is as follows: Carbon, 0.95 to 1.15%; manganese, 10 to 13%; silicon, 0.20 to 0.40%; phosphorus, under 0.10%; sulphur, under 0.06%.

AN IRON ORE CEMENT of a new constituency has been patented by Messrs. Arthur C. Spencer and Edwin C. Eckel. As heretofore manufactured these hydraulic cements in which iron is substituted wholly or in part for the alumina ordinarily contained therein, have obtained their iron from iron oxides, comparatively rich in iron, but of too great value in other industries to make their use a commercial economy in iron-cement. The new invention proposes to use for the iron element, in the words of the patent, "a cheap natural and easily fusible silicate of iron and potassium called glaucorite or the related mineral greenolite, both of which have the common name greensand; or rocks containing greensand." Provision is also made for the use of any natural material containing a sufficiency of iron silicate. It is claimed that these silicates are not only superior to the oxides on account of their cheapness, natural fineness of grain, accessibility, ready fusibility and lack of value for other purposes, but that in greensand, iron is already chemically combined with silica, which is needed in the finished cement and the reaction involved in sintering therefore takes place at a lower temperature than is possible if iron be in the form of an oxide.

Iron-ore cement in experiments and the short time actual use which has been possible, has shown up remarkably well in sea water, so that, in Germany especially, recommendations are frequently made to require its use in all ocean concrete structures. A description of its properties appeared in Engineering News, Dec. 12, 1907, p. 646.

THE TARIFF ON ASPHALT was discussed at some length in a communication recently sent to the Chairman of the Ways and Means Committee, U. S. House of Representatives, by Mr. J. W. Howard, Consulting Engineer, 1 Broadway, New York City. A portion of Mr. Howard's letter follows:

The tariff act of 1907 provides a duty of $1.50 per ton on crude asphalt and $3 per ton on refined asphalt.

This should be modified by a reduction on crude and a slight change in the rate and manner of applying it to refined asphalt, to have it apply to the pure bitumen or pure asphaltum content in the various refined asphalts and their compounds imported. It is the asphaltum or bitumen which is of value in such importations, not the various foreign substances found in different proportions in different asphalts.

Crude asphalt and crude bitumen and their natural compounds should be returned to the free list, being admitted free of duty; but any and all of said substances, if refined or advanced in value by artificial or special treatment, should pay a duty of $5.60 per ton (2,000 lbs.) of pure asphaltum or pure bitumen content.

This is the differential in cost of production, labor, mining, refining in the United States with a fair profit, compared with like foreign materials. This also takes into account the average transportation charges within the United States for domestic asphalts, from many points where now produced in the United States (California, Utah, Colorado, Oklahoma, Kansas, Kentucky, Texas, etc.) to several hundred cities where such domestic products compete with imported asphalts; such imported asphalt all arriving on the Atlantic Seaboard and not being able to penetrate far because of westbound freights.

In addition, Mr. Howard discussed other phases of the subject and summarized the various asphalt deposits now drawn upon for the asphalt used in the United States, grouping these deposits to correspond with the interests controlling them, which he named.

Personals.

Mr. G. L. Moore has been appointed Engineer of Maintenance of Way of the Lehigh Valley R. R., to succeed Mr. R. G. Kenly, resigned.

Mr. Robert Burgess has become Chief Engineer of the Honduras National R. R. His headquarters are at Truxillo, Honduras, C. A.

Mr. J. C. Vining, General Superintendent of the Colorado Midland Ry., has resigned and will become President of a cold storage company which he has organized in Grand Valley, Colo.

Mr. David S. Hawkins has opened an office as a consulting engineer at 1021 Rose Bldg., Cleveland, Ohio. He will make a specialty of legal consultation and the elimination of losses in foundry operation.

Mr. Adrian Iselin, Jr., Vice-President of the Buffalo, Rochester & Pittsburg Ry., has been elected President of that line to succeed Mr. Arthur G. Yates, deceased. Mr. W. T. Noonan, General Manager, has been made Vice-President, but will retain also his previous position and title.

Mr. Robert Earl has been nominated by Governor Hughes as Democratic member of the New York State Highway Commission. Mr. Earl was until recently cashier of the Herkimer (N. Y.) National Bank and was at one time a farmer. Reports quote him as saying he has never had any experience in the planning or construction of roads.

Mr. Arthur P. Davis, Chief Engineer of the Reclamation Service, and Mr. D. C. Henry, Consulting Engineer of the Service, sailed from New York for Porto Rico on Feb. 20. The Legislature of Porto Rico has provided for the raising of $3,000,000 to be expended on irrigation works, the plans for which are to be approved by the Reclamation Service engineers. Mr. B. M. Hall, of the Reclamation Service, went to Porto Rico some time ago.

Obituary.

Samuel H. Chittenden, a retired civil engineer, died recently at the home of his brother, Mr. Geo. B. Chittenden, in Washington, D. C. He was born in Madison, Conn., in 1845, and graduated in 1867 at the Yale Sheffield Scientific School. In 1868 he was employed by the Union Pacific R. R. on survey work west of Cheyenne, Wyo., and afterward was engaged in similar work in Alabama and Georgia. He had also some experience in mining in Arizona and New Mexico and eventually settled in Washington, D. C., as a contracting engineer. He retired from active work in 1890 and has resided since that time at East River, Conn.

Charles J. MacConnell, Chief Engineer, U. S. N., retired, died at his home in Brooklyn, N. Y., Feb. 16. Mr. MacConnell was born in Pennsylvania in 1837 and had considerable experience as a mechanical and civil engineer before his entry into the service in 1861. His education was received at the State Normal School in Trenton, N. J. He was an Assistant Engineer of Admiral Farragut's fleet at the capture of New Orleans and, in the war with Spain, was Fleet Engineer of the North Atlantic Squadron during the blockade of Havana Harbor. Mr. MacConnell became a Chief Engineer in 1885 and, at the time of his retirement in January, 1899, attained the rank of Rear Admiral. Since his retirement he has practiced as a consulting engineer at his office in New York City.

Carroll D. Wright, President of Clark College at Worcester, Mass., since 1902,, and of great prominence as an economist and statistician, died Feb. 20 in that city. Mr. Wright was born at Dunbarton, N. H., in 1840. At the time of his death he held the degrees of A. M. and LL. D. from Tufts College, Ph. D. from Dartmouth and LL. D. from Wesleyan, Clark and Amherst. He served with the 14th New Hampshire Volunteers during the Civil War, rising from the rank of private to that of colonel. In 1872-3 he was a member of the Massachusetts Senate and revealed his talent as a statistician with the result that he was placed in charge the following year of the Massachusetts Bureau of Statistics of Labor, remaining in this position until 1888. From 1885 to 1905, he was U. S. Commissioner of Labor, and in 1893-7 he was in charge of the completion of the eleventh U. S. Census. He was appointed by the President, in 1902, a member of the U. S. Anthracite Strike Commission. Mr. Wright was the author of a number of books on social, statistical and economic subjects. He was a member of the board of trustees of the Carnegie Institution at Washington and was a Past-President of the American Association for the Advancement of Science and the American Statistical Association.

Edwin Reynolds, Past President of the American Society of Mechanical Engineers and Consulting Engineer of the Allis-Chalmers Co., died Feb. 19 in Milwaukee, Wis. Mr. Reynolds was born at Mansfield, Conn., in 1831 and was educated in the schools of that town. At the age of 16 he began a three years' apprenticeship in a machine shop and after completing this term he continued as a journeyman machinist in various employs until, in 1857, he became Superintendent of Shops for Stedman & Co., at Aurora, Ind. In 1861 he returned to the East and entered the employ of the Corliss Steam Engine Co., of Providence, R. I., of which he became General Superintendent in 1871. He resigned this position in 1877 to accept a similar one with E. P. Allis & Co., at Milwaukee. It was here that Mr. Reynold's engineering ability reached its highest development. Soon after going to Milwaukee, he designed the Reynolds-Corliss engine, which notable achievement marked the beginning of the firm's commercial success. He later introduced the first triple-expansion pumping engine, the cross compound hoisting engine for mining work and many other original designs contributing to the development of the steam engine. Prominent among the more recent examples of his work are the enormous horizontal-vertical compound units designed for the Manhattan Ry. power house in New York City. A comprehensive account of Mr. Reynolds' life was published in Eng. News, Jan. 16, 1902, p. 43, on the occasion of his election to the presidency of the American Society of Mechanical Engineers.

Engineering Societie

COMING MEETINGS.

NORTHWESTERN CEMENT PRODU ASSOCIATION.
March 2-4. Annual convention at M polis, Minn. Secy., J. C. Van Doorn, 836 Secur Bank Bldg., Minneapolis, Minn.

INTERNATIONAL MASTER BOILER M ERS' ASSOCIATION.
April 27 to 30. Annual meeting at isville, Ky. Secy., H. D. Vought, 95 Liberty St., York City.

RAILWAY SIGNAL ASSOCIATION.
March 15. At Chicago, Ill. Secy., C. Rosenberg, Times Bldg., Bethlehem, Pa.

AMERICAN RAILWAY ENGINEERING AND MAIN. TENANCE OF WAY ASSOCIATION.
March 16-18. Annual convention at Chi Ill. Secy., E. H. Fritch, 962 Monadnock Block, Chicago.

AIR BRAKE ASSOCIATION.
May 11-14. Annual meeting at Richmo Va. Secy., F. M. Nellis, 53 State St., Boston, Ma

ENGINEERS CLUB OF PHILADELPHIA.—At the meeting held on Feb. 6, Mr. T. J. Little, presented a paper on "Recent Improvements in Gas-Lighting Apparatus."

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.—At the meeting held on Feb. 19 Dr. Charles P. Steinmetz presented a paper entitled "Prime Movers," which was printed in the February "Proceedings," pp. 135-156.

MUNICIPAL ENGINEERS OF THE CITY OF NEW YORK.—At the regular meeting of Feb. 24 a paper entitled "An Additive Method of Run-Off Determination for Storm Water Sewers" was presented by Mr. Carl H. Nordell.

NEW YORK ELECTRICAL SOCIETY.—At the meeting held in the Engineering Societies' Building, Feb. 17, Dr. Louis Bell, M. Am. Inst. E. E., delivered a lecture on 'Modern Practice in Street Lighting." A number of lantern slides were shown illustrating prevalent European street lighting practice.

AMERICAN FOUNDRYMEN'S ASSOCIATION.—In connection with the annual convention at Cincinnati, Ohio, May 17-22, will be held the exhibition of working foundry machinery and equipment prepared by the Foundry & Manufacturers' Supply Association. Formal meetings well be held on May 18, 19 and 20.

APPALACHIAN ENGINEERING ASSOCIATION.—At the annual meeting in Morgantown, W. Va., Feb. 6, officers were elected as follows: President, E. A. Schubert, Roanoke, Va.; Vice-President, Baird Halberstadt, Pottsville, Pa.; Secretary, Henry M. Payne, Morgantown, W. Va.; Treasurer, C. E. Krebs, Kanawha Falls, W. Va.

WASHINGTON SOCIETY OF ENGINEERS.—At the meeting held Feb. 16, Mr. Van H. Manning, Topographer of the U. S. Geological Survey, gave an illustrated lecture outlining the reclamation work along the Mississippi River extending south from a point ten miles south of Memphis and involving an area of 7,000 square miles. An informal reception was given at this meeting to the seven engineers who accompanied President-elect Taft to the Panama Canal Zone.

RAILWAY CLUB OF WESTERN CANADA.—The headquarters of this newly organized society will be in Winnipeg, Man. Meetings will be held on the second Monday of each month excepting in June, July and August. At the inaugural meeting, officers were elected as follows: Honorary President, William Whyte; Honorary Vice-Presidents, M. H. McLeod, G. J. Bury, G. W. Caye, Wilford Phillips; President, Grant Hall; First Vice-President, L. B. Merriam; Treasurer, T. Humphries; Secretary, W. H. Roseberry.

OHIO ELECTRIC LIGHT ASSOCIATION. The 15th annual convention will be held in Toledo, Ohio, July 13-15. Annual conventions have formerly been held in August. The program will include the following topics: The Titanium Arc Lamp, Reports and Discussion on Tungsten Lamp Experience, The Supply of Current, for Lighting, to Other Towns from a Centrally Located Station; Report on Methods of Lamp Renewals; Commercial Organization of an Electric Light Co. and Its Relation to the Public; Factors Determining the Price to be Charged for Street Lighting.

AMERICAN SOCIETY OF INSPECTORS OF PLUMBING AND SANITARY ENGINEERS.—The fourth annual meeting was held at Omaha, Neb., Feb. 8-10. Among the papers presented were the following: "The Cesspool and Its Dangers," Burton J. Ashley, Chicago; "Sewage Disposal in Fresno, California," Robt. H. Ellithorpe, Fresno; "Siphons for Sewage Disposal Tanks," Alfred R. McGonegal, Washington, D. C.; "Sanitation on the Panama Canal Work," Chas. L. Stockelberg, Inspector of Plumbing, Culebra, Canal Zone; "A Standard for Plumbing Material," E. Willard, Kokomo, Ind.; "Water Waste," Henry B. Davis, Washington, D. C.